SKELETON CONSTRUCTION IN BUILDINGS.

WITH

NUMEROUS PRACTICAL ILLUSTRATIONS OF HIGH BUILDINGS.

BY

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PREFACE.

THE author has endeavored in this volume to describe and illustrate the method of skeleton-constructed buildings; a new type of structure, which calls for principles entirely different from the old system of cast-iron fronts and cast-dowelled columns with wooden girders.

He has been induced to prepare it from the fact that the improvements in modern iron construction, especially in high buildings, has been so rapid during the past few years that no work, however recent, meets the latest requirements.

Notwithstanding the fact that the subject of the strength of columns has been ably treated of again and again, tables of tests of various-shaped columns are given, and in a few chapters especial stress has been laid upon the advantages and disadvantages of different shapes of iron and steel in columns for making rigid connections with the floor beams, curtain-wall girders and with each other—a necessary requirement, indispensable to good construction. Other chapters are devoted to the details and calculations attending the erection of high buildings using the skeleton construction,—the system of wind-bracing, curtain-wall supports, and foundations.

WM. H. BIRKMIRE.

NEW YORK, April, 1893.

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SKELETON CONSTRUCTION IN BUILDINGS.

CHAPTER I.

GENERAL AND DESCRIPTIVE.

THE method of skeleton construction has been developed by the use of iron and steel in the erection of fire-proof buildings, and it seems to have solved the problem of economizing space in the lower floors of high and narrow buildings. In the ordinary methods of building, the higher the wall the thicker it must be at its lower parts, but the lower stories are the most valuable; yet it is in these that the greatest area of a valuable lot must be surrendered to enormously thick walls. Therefore, every foot gained on the inside measurements increases the availibility of the structure.

In the buildings where the skeleton construction is used throughout, heavy masonry walls are not known, and what appears to be such in the finished state is simply a veneer of some fire-proof material, yet the frame of the building is not a mere heap of beams and columns; but as one example after another is erected we find that the details and connections are being carefuly studied, and the whole braced and anchored so completely that the metal construction may be raised hundreds of feet from foundation to roof without the aid of any masonry—a great metal structure, strong in its own strength, not only to carry the direct loads which may be placed upon

it, but also to resist all lateral strains to which it may be subjected.

The architectural appearance of our large cities is being rapidly altered by this new system. It imposes no new conditions on the architect, except as to the engineering of the metal frame; and in a few years, with the skill already displayed in treating these problems, many new designs will be brought forth, notwithstanding the great height to which they are built.

The skeleton construction consists in the use of cast-iron, wrought-iron, or wrought-steel columns in the side-walls, connected longitudinally at the floor levels with beams, lattice or compound riveted girders supporting the thin curtain walls 12 to 20 inches in thickness; in addition the weight of the floors are transmitted to the longitudinal girders and columns, so that the latter support the entire building.

The thin curtain walls are generally built of brick, and extend from the top of any wall girder to the underside of the next story girders, extending a sufficient distance outside to cover the girders and columns with masonry, and continue in this manner to the top of the building.

The New York building law imposes certain conditions upon the use of the curtain walls, in that the curtain walls of the lower stories shall be built thicker. Why this is so the author has not been able to determine, as a 12-inch wall from foundation to roof resting upon these wall girders would seem sufficient for all purposes.

There are some variations in the use of the skeleton frame which will also be fully illustrated and described in the pages to follow. In some cases the frame work of columns and girders are carried up to within four or five stories of the roof; then continuous girders are placed upon the top of columns to support the upper stories of masonry walls.

In some cases the columns begin at the base course on stone, iron, or steel beams.

In others from the top of foundation-walls level with the sidewalk or curb level.

Another variation is where the walls and columns are separated, the walls built heavy enough to carry their own weight, and the columns support the floor and their loads.

Then again the longitudinal girders may be placed in every second floor, and the walls are made 20 to 24 inches in thickness below the fourth or fifth tier.

When the first examples of the skeleton construction were completed many questions were raised, and no doubt there exists in the minds of many at the present time that the greater expansion of one material over another might work some trouble. Events have proven that the temperature of this climate from the greatest cold to the greatest heat exerts no appreciable effect, especially while the metal is covered with masonry or some fire-proof material.

Representative Chicago High Buildings.—Very many journals have been devoting much space to descriptions of the steel skeleton type of buildings, more especially those of Chicago, and the majority of writers call it the "Chicago Steel Skeleton Construction."

There can be no doubt that Chicago's business districts has undergone a remarkable transformation within a few years, as any one who visits that city to-day would scarcely believe that, something like eight years ago, the tallest buildings were not ever eight stories high.

Immediately after the Great Fire that totally ruined the business district, the city was built up hurriedly; many handsome buildings were erected, and these have been torn down to give way to very high structures, which command much admiration throughout the country.

One of the first of the many tall buildings erected was the Montauk Block, which stands on Monroe Street, just west of Dearborn. It was built about eight years ago, and is ten

stories high. When contrasted with some of the latest structures it is comparatively insignificant.

The Woman's Christian Temperance Union Building, as shown in the above engraving, Fig. 1, popularly called the

FIG. 1.

Woman's Temple, because it was built by the ladies of that organization from contributions raised in small sums in all parts of the country, is perhaps the handsomest big building in Chicago. Its lines are so proportioned that its enormous height rarely elicits comment.

Then, again, the Owing's Building, Fig. 2, situated on Dearborn and Adams Streets, on account of its immense height, as contrasted with its slim frontage, is one of the notable structures of Chicago. The German Opera House, Fig. 3, is another of the striking buildings in that city. Chicago may

FIG. 2.—THE OWING'S BUILDING, CHICAGO.

not count more tall buildings than in other cities, but she has, no doubt, a greater number at present of "sky-scrapers" as they are called in the West, within a given area.

The Masonic Temple is regarded as one of the greatest achievements of high building construction and engineering

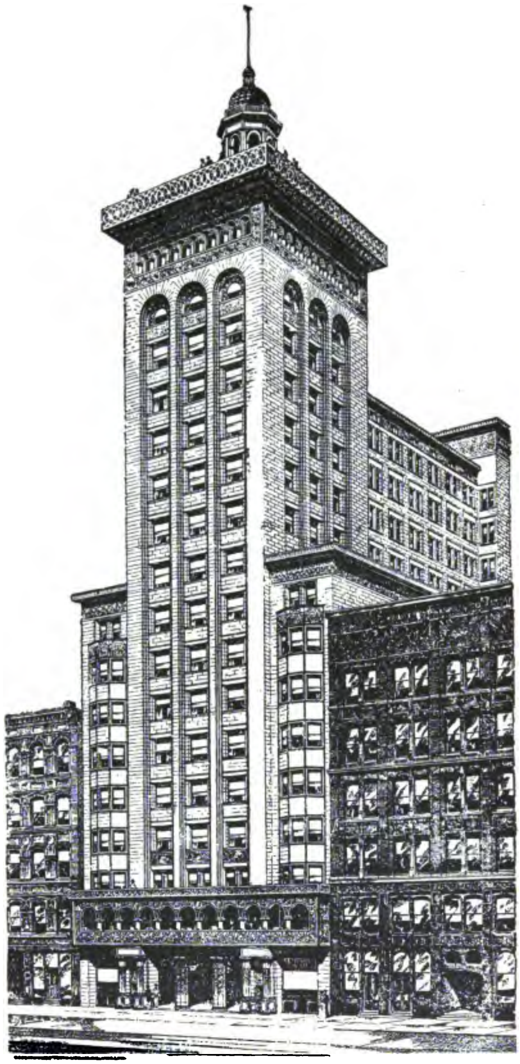


FIG 3.—GERMAN OPERA HOUSE, CHICAGO, ILL. ADLER & SULLIVEN, ARCHITECTS.

(see perspective, Fig. 4).* It is situated at the corner of State and Randolph Streets, and designed by Messrs. Burnham and Root, architects of Chicago. The building extends 170 feet on State Street to a 40 feet alley, and 113 feet on Randolph Street to a 25 feet alley. The height from the sidewalk to the top of coping is 274 feet. The building is 20 stories, and contains 5,436,000 cubic feet exclusive of the court.

The street fronts are of dressed granite up to the sills of the fourth-story windows; above that of terra cotta and brick, of a gray and mottled color to match the granite. There are fourteen passenger elevators arranged in a circular curve at the rear of the main entrance. There are also freight elevators. All balconies have floors and soffits of marble and mosaic. Hall columns in the court are covered with alabaster. All interior metal is of bronze finish, highly ornamented. The inside court is lined of marble throughout. On the roof is a promenade deck, 100 × 120 feet, covered with a skylight and enclosed with glass. To the top of this skylight from the sidewalk is 302′ 1″. All piers have steel columns inside which carry all the floor loads. With the exception of six piers, the whole building above the fourth floor is carried on the columns. Two systems of vertical bracing run through the narrow way of the building from top to bottom, one each side of the elevators. These rods run through two floors and cross one column. Each of the above columns or pair of columns, as mentioned, are provided with independent footings, which reduces and distributes the pressure uniformly on the soft and treacherous soil. The architects of Chicago probably have to deal with the most unfavorable conditions for securing a good foundation for these heavy buildings.

The soil under the business district consists of a black loamy clay, which is somewhat firm at the surface, but a few feet below the surface the soil becomes quite soft, growing

* For a fuller description, see the *Engineering Record*, Jan. 21, 1893.

more so the deeper the excavation is carried. The first of the large structures were built with continuous foundation walls with wide footings. This method did not prove suc-

FIG. 4.—MASONIC TEMPLE, CHICAGO, ILL. BURNHAM & ROOT, ARCHITECTS.
(From Architecture and Building.)

cessful. After many experiments, the foundations were divided into isolated piers, the footing being carefully proportioned, according to the load upon it, so that all should settle at exactly the same rate, without any detriment to the superstructure.

The footings of the piers in the Masonic Building, as in the majority of Chicago buildings, are built of steel rails and concrete, and crossed three or four times, thus insuring a great spreading in a small height.

Under a single column of the Masonic Building the concrete is $15' 2'' \times 15' 2''$. On top of this 18 steel rails were laid; then 18 at right angles to these; then 10 parallel to the lower 18 and 10 parallel to the upper 18, making a total of 56 pieces.

The old Board of Trade Building, at the corner of La Salle and Dearborn Streets, lately remodelled, presents a front remarkable even in the city of tall buildings. From a seven-story structure of rather inferior design it has been remodelled into a fourteen-story palace.

Down on Dearborn Street the Manhattan Block is an immense structure, eighteen stories high. The Tacoma Building at Madison and La Salle, was considered a high building a few years ago; it is now overshadowed by a number of much taller buildings.

Representative New York High Buildings.—Considering the present rapid development of the skeleton construction and this necessity for high buildings, New York City takes its place at the head, not only in the designs, but in the details of the construction. And where other cities have confined their high structures within narrow limits, it is not so in New York; they are scattered along the principal thoroughfares from the Battery to Central Park, where high buildings are quite common.

Among the many notable and handsome structures adjoining the Park are the New Netherlands, a hotel built in 1892, by W. W. Astor. It occupies a site 100 feet by 125, on the corner of Fifth Avenue and Fifty-ninth Street, and has a cellar and basement below the street level, and seventeen stories above, the four upper stories being in the picturesque high roof. Nine hundred steel columns and about 4500 steel beams were used in the construction of this building, the

details of which, and other parts of the building construction are further explained and illustrated in Chapter VIII of this

FIG. 5.—PROPOSED OFFICE BUILDING FOR THE N. Y. SUN, PARK ROW.

volume. The Savoy is another of the tall buildings in the same locality, situated at the southeast corner of Fifth Avenue

and Fifty-ninth Street, used as a hotel. It measures 75 × 150 feet, and has an extension of 100 feet more at the rear. It is an eleven-story steel frame structure, faced with Indiana limestone, in the Italian Renaissance style of architecture. It was opened in 1892.

The Plaza Hotel, directly opposite the Savoy, faces the Plaza at the Fifth Avenue and Fifty-ninth Street entrance to Central Park, overlooking the main Park entrance.

The Hotel Majestic is another of these high and elegant hotels built in this locality.

There are also a number of lofty and extensive apartment houses in the vicinity of Central Park. One of the largest is the Dakota, at Central Park West and Seventy-second Street. It is a many-gabled building, in the style of a French château.

Then in Fifty-ninth Street near Seventh Avenue, south side of the Park, are the Central Park or Navarro Flats, which include several independent houses constructed as a single building. The different houses in the group are known as the Madrid, Granada, Lisbon, Cordova, Barcelona, Valencia, Salamanca, and Tolosa, all combined, with numerous balconies and façades, in the Spanish style.

The Osborne, Seventh Avenue and Fifty-eighth Street, is another tall structure of the above class.

At the extreme southern part of the city the greatest number of high buildings are used as offices. One of the finest and largest is the Washington Building, at the foot of Broadway, overlooking Battery Park and the Harbor. The Washington Building was completed in 1884. It covers 17,000 square feet of land, and is thirteen stories in height, the two upper stories being in the mansard roof.

Between the central and lower portions of the city a few of the highest buildings erected and about to be erected are shown by a few plates, such as the World Building. Fig. 6, erected in 1889–90, is the tallest office building known, reaching 309 feet from sidewalk to lantern; or 375½ feet from the

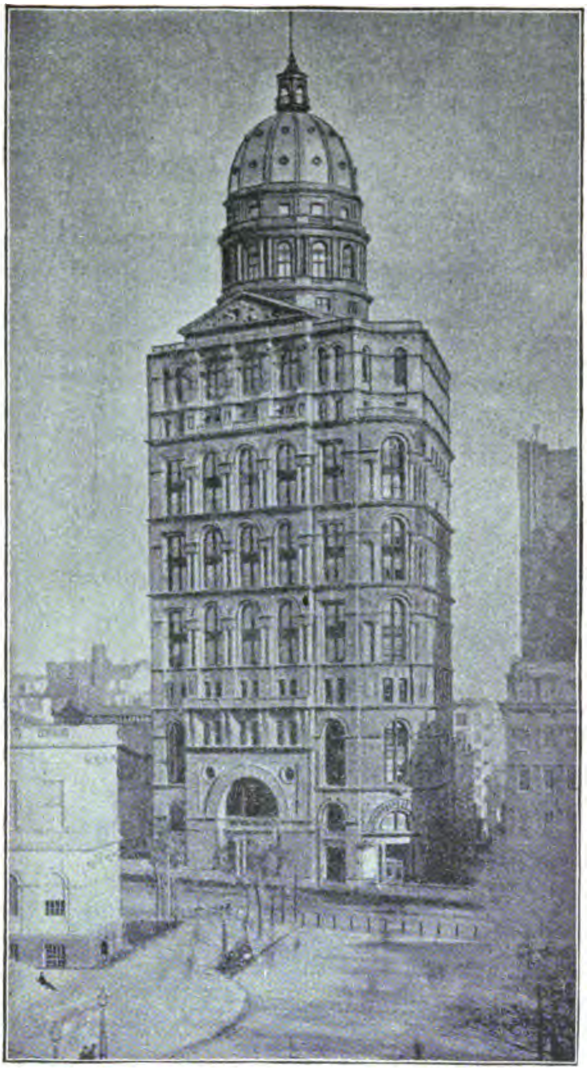


FIG. 6.—THE WORLD BUILDING, PARK ROW, FACING CITY HALL PARK, N. Y
George B. Post, Architect.

bottom of foundation to the top of the flagstaff. It has a huge skeleton of iron and steel sustaining its twenty-six stories.

The Manhattan Life Insurance Company is preparing to erect a building at Nos. 64 and 68, Broadway, which will surpass the World Building in height—a view of which is shown in Fig. 7. The building is sixteen stories above the sidewalk. Then comes the seventeenth story, 14 feet; the eighteenth, 26 feet; the nineteenth, 23 feet; the twentieth, at the floor of the lantern, 27 feet, making a total of 326 feet from the sidewalk. In style it will be a valuable contribution to the architecture of lower Broadway, and will make an imposing appearance among its stately neighbors; the Standard Oil Company, the Columbia Building, Aldrich Court, the Consolidated Stock and Petroleum Exchange, the Union Trust Company, and even the tall spire of the Trinity Church will be thrown in the shade.

The proposed office building of the New York *Sun*, situated on Park Row opposite City Hall Park, as designed by Bruce Price, architeet, and shown by the sketch Fig. 5, contemplates a building thirty-two stories in height, and if carried out as the architect intends will, no doubt, take its place as one of the handsomest office structures in existence.

All the above buildings are not, strictly speaking, of the skeleton type; but a number of the principal ones using this style of construction are further described and detailed in the following pages, such as the Havemeyer Building, Postal Telegraph, the Home Life Insurance Company, the Waldorf, the Western Union Annex, etc.

New York Building Law in Relation to Skeleton Construction.—For the skeleton construction, the existing law, passed April, 1892, makes some provision: "Curtain walls of brick built in between iron or steel columns, and supported wholly or in part on iron or steel girders, shall not be less than twelve inches thick for fifty feet of the uppermost height

FIG. 7.—MANHATTAN LIFE INS. CO. BUILDING, 64 & 68 BROADWAY, N. Y.
Kimball & Thompson, Architects.

thereof, or to the nearest tiers of beams to that measurement in any building so constructed; and every lower section of fifty feet or to the nearest tier of beams to such vertical measurement, or part thereof, shall have a thickness of four inches more than is required for the section next above it down to the tier of beams nearest to the curb-level; and thence downwardly the thickness of walls shall increase in the ratio prescribed in section 474 of this title for the thickness of foundation-walls.

Curtain-walls may be four inches less in thickness than is specified respectively for walls of dwellings and buildings, but no curtain-wall shall be less than twelve inches thick.

Section 474.—Foundation walls shall be construed to include all walls and piers built below the curb-level or nearest tier of beams to the curb, to serve as supports for walls, piers, columns, girders, posts, or beams. Foundation-walls shall be of stone or brick. If built of stone they shall be at least eight inches thicker than the wall next above them to a depth of twelve feet below the curb-level; and for every additional ten feet or part thereof deeper, they shall be increased four inches in thickness. If built of brick they shall be at least four inches thicker than the wall next above them to a depth of twelve feet below the curb-level, and for every additional ten feet, or part thereof deeper, they shall be increased four inches in thickness.

The footing or base course shall be of stone or concrete or both, or of concrete and stepped up brick-work, of sufficient thickness and area to safely bear the weight to be imposed thereon; if the footing or base course be of concrete, the concrete shall not be less than twelve inches thick; if of stone, the stones shall not be less than two by three feet, and at least eight inches in thickness for walls and at least twelve inches wider than the bottom width of said walls, and not less than ten inches in thickness if under piers, columns, or posts, and at

least twelve inches wider on all sides than the bottom width of said piers, columns, or posts.

Section 485.—Where columns are used to support iron or steel girders carrying *curtain-walls*, the said columns shall be of *cast-iron*, *wrought-iron*, or *rolled steel*, and on their exposed outer and inner surfaces be constructed to resist fire by having a casing of brick-work not less than four inches in thickness and bonded into the brick-work of the curtain-walls, or the inside surfaces of the said columns may be covered with an *outer shell* of iron having an air space between; and the exposed sides of the iron or steel girders shall also be similarly covered in and tied and bonded.

When the thickness of the curtain-walls is twelve inches, the girders for the support of same shall be placed at the floor line of each story, commencing at the line where the thickness of twelve inches starts from, and when the thickness of such walls is sixteen inches the girders shall be placed not farther apart than *every other* story, at the floor line commencing at the line where the thickness of sixteen inches starts from, provided that at the intermediate floor line a suitable tie of iron or steel shall rigidly connect the columns together horizontally, and that the ends of the floor-beams do not rest upon the said sixteen-inch walls.

When the *curtain-walls* are twenty inches or more in thickness and rest directly on the foundation-walls, the ends of the floor-beams may be placed directly thereon, but at or near the floor line of each story ties of iron or steel encased in the brick-work shall rigidly connect the columns together horizontally.

CHAPTER II.

COLUMNS.

Columns.—The first examples of the skeleton construction in buildings were those erected with *cast-iron* columns. Cast-iron at the time of their erection, and no doubt is at the present time, produced more quickly and cheaper than wrought-iron or steel columns, and these were two very important factors in the problem.

The constructors and producers of cast-iron advocate its use only as the material for the columns inclosed in the walls. They claim also that the oxide of iron paint so commonly used for coating iron soon dries out, leaving a coating of dry, broken scale or powder. Between the columns and the outer air are only a few inches of brick or some fire-proof material, through which dampness soon finds its way. In wrought-iron, they claim that rust honeycombs and eats entirely through the metal

Mild steel rusts faster than wrought-iron at first, then slower. Cast-iron, on the contrary, slowly oxidizes in damp situations; rust does not scale from it, and the oxidation when formed is of much less dangerous kind, extending only a little way into the metal to about the thickness of a knife-blade, and then stops for good. Cast-iron of goodly thickness offers a far better resistance to fire, or fire and water combined, than wrought-iron or steel.

The experiments undertaken by Prof. Bauschinger, of

Munich, in reference to the safety of cast-iron columns when exposed to the action of great heat are quoted. "Having arranged some cast and wrought iron columns heavily loaded, exactly as they would be if supporting a building, had them gradually heated; first, to three hundred degrees, next to six hundred, and finally to red heat, then suddenly cooling them by a jet of water, just as might happen when water is applied to extinguish a fire.

"The experiment showed that the cast-iron columns, although they were bent by the extreme heat and exhibited transverse cracks when cold water was applied, yet they supported the weight resting on them; while the wrought-iron columns were bent before arriving at the red heat, and were afterwards so much distorted by the water that the restraightening them was out of the question; in fact, if supporting a real building, they would no doubt have utterly collapsed under the weight they had to sustain."

If the brick-work or fire-proofing which surrounds the wall or interior columns can be depended upon as a protection for the metal against the effects of fire and water, the above experiment would lose its weight against the use of wrought-iron or steel.

The objection to wrought-iron or steel on account of rusting may seem more real, and yet we have seen pieces of wrought-iron beams, anchors, etc., taken from very old walls unharmed by rust.

There is, however, considerable distrust of cast-iron in *high and narrow building*, especially in relation to the connections with the floor and wall girders. Brackets and lugs are apt to break suddenly and completely, but with wrought-iron and steel will bend a great deal without breaking, and that rivets are stronger than bolts. To this objection it can be said that the brackets and lugs, instead of being cast with the columns, can be put on with angle-knee connections, drilled holes in the columns and with any number of bolts, which in a great many

of our high buildings has proven entirely satisfactory, *where lateral bracing is not required.*

The advocates of wrought-iron and steel columns claim that cast-iron is rigid and unyielding, and that its coefficient of elasticity is much lower than that of wrought-iron or steel, and the cast-iron column is not as stiff as the others, and will not on the whole produce as rigid and unyielding a structure.

Where high and narrow buildings are concerned, much attention is given to the bracing against wind forces—that is, in the stiffness of the joints and the stability of the structure upon the foundation, and when the bracing is a portion of the frame construction, the difficulty of doing it properly with cast-iron columns is very great, but with wrought-iron or steel these difficulties are largely removed.

It is not the intention of the author to enter into any discussion on the question of which should be adopted, but to confine the subject to what has already been done, and illustrate the practices of the present time, especially the buildings he has closely followed in having charge of the constructive details at the Architectural Iron Works. We have very many handsome buildings constructed where cast-iron, wrought-iron, and steel columns have been exclusively adopted, each system will no doubt be copied for years to come unless some radical change will turn the tide into another channel.

But we are entering on an age of steel. Rolling mills produce it quicker and cheaper than any other metal, and the change from cast-iron to steel for columns, beams, and girders, especially in our large buildings, has been generally adopted by the prominent architects throughout the country. One after another the advocates of cast-iron have fallen into line in favor of wrought-iron and steel in high and narrow buildings, but for buildings with a large base cast-iron will continue to be popular.

Cast-iron columns have been used, and are still extensively

adopted, in some of our noted buildings. In New York we have in course of construction, together with those already built:

Postal Telegraph Building	D., L. &. W. R. R. Building
Decker Bros. Building	The Western Union Annex
The Waldorf	Lincoln Building
Jackson Building	McIntyre Building
Scott & Bowne Building	Mutual Life Annex (wall col's).

In Chicago cast-iron columns have been used in such buildings as

The Rookery	The Auditorium
Home Insurance Building	The Chamber of Commerce
The Monon Block	Manhattan Building
Western Bank Note Bldg.	Unity Building
Tacoma Building	Owens Building.
Cold Storage Building	

Wrought-iron and steel columns have been used in New York in the following buildings:

The New Netherlands	Home Life Ins. Co. Building
Havemeyer Building	Hotel Majestic
Lancashire Building	Mail and Express
World Building	Mutual Reserve Fund Building
etc.	etc.

In Chicago a few of the noted wrought-iron and steel structures are:

Rand McNally Building	Masonic Temple
The Ashland Block	German Theatre
Venetian Building	The Pontiac
The Kearsarge	Northern Hotel
The Fair	Woman's Temple, etc

Many noted buildings in all the other large cities have used each system.

The heights of buildings using in cast-iron compare favor-

ably with those using wrought-iron and steel—a few of which are compared below:

Cast-iron Structures.		Feet.	Stories.	Steel Structures.		Feet.	Stories.
Chicago,	Rookery.........	164	12	Chicago,	Northern Hotel,	168	14
N. Y.,	Postal Telegraph .		14	"	Masonic Temple,	254	20
Chicago,	Unity Building...	210	17	N. Y.,	New Netherlands,	217	17
"	Tacoma Building,	165	13	"	Home Life.......		
"	Manhattan	210	16	"	Havemeyer......	175	15

Cast-iron Columns are usually made hollow round, Fig. 13, or when built in walls, as in the skeleton frame, hollow square, Fig. 10. Some of the variations brought about by the skeleton frame are shown, as Fig. 8, or what is called the H-shape, with an open web, and Fig. 9, similar to Fig. 8, but with a solid web.

Then again we have the modification of the square column, as in Fig. 11. The side or back adjoining the party wall is

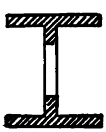


Fig. 8.

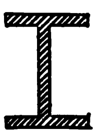


Fig. 9.

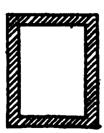


Fig. 10.

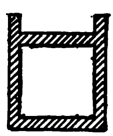


Fig. 11.

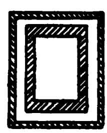


Fig. 12.

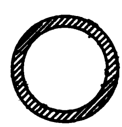


Fig. 13.

moved nearer the axis of the column, so that a greater distance could be had for fire-proofing the body of the columns. This section was used in the McIntyre Building, Broadway and Eighteenth Street, New York, and changed from the shape Fig. 10 to this by the architect, and approved by the Building Department.

The square column encased with a shell, as Fig. 12, was used in one of the first buildings adopting the skeleton frame.

In making a selection from the different shapes for the skeleton structure, it is very important to adopt the form that presents the smoothest or unbroken surface for connections with the floor and curtain-wall girders.

It is also important that the masonry should be built

solidly around the columns. The H-sections, Figs. 8 and 9, are no doubt better adapted for this purpose than the rectangular shapes, Figs. 10, 11, and 12.

The hollow circular column, Fig. 13, is the least desirable for building in with the walls, more difficult to fire-proof, but in connecting with the girders that portion of the column can be cast square.

Wrought-iron and Steel Columns as used for the skeleton frame are various, and when a compound column section is required; very many rolled shapes can be riveted together to make up the required section. Those made up of Z-bars and a single web plate, as Fig. 14, are about the simplest form of riveted columns.

FIG. 14.

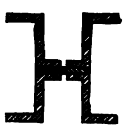


FIG. 15.

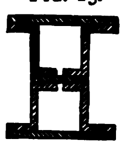


FIG. 16.

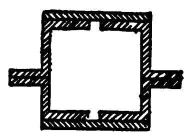
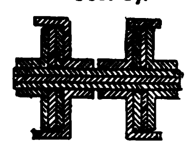


FIG. 17.



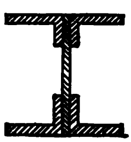


FIG. 18.

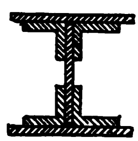


FIG. 19.

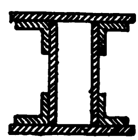


FIG. 20.

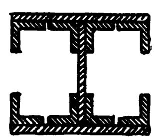


FIG. 21.

The section is increased by the use of cover plates riveted to the outer leg of the Z's and shown at Fig. 15. Then again, we have the rectangular section of Z's, Fig. 16. The section shown by Fig. 17 is a Z-bar column used in the Venetian Building, Chicago, a twelve-story structure; the additional required area being made up of plates and angles. The column is $13\frac{1}{4}'' \times 21'' \times 27'.1\frac{1}{4}''$ long.

It is a well-known fact that metal near the neutral axis of a column is good for little, and that the capacity of equal areas varies as the metal is removed from the neutral axis.

It seems, therefore, that a better proportioned column

could be adopted for the requirement of the case. In Fig. 18 the column section is made up of four angles and a single web, and in Fig. 19 a single plate is added or a number of plates to make up the required section.

Then the rectangular shape, Fig. 20, made up of angles and plates, requiring eight lines of rivets.

The author has not been able to find a section like that of Fig. 21, used in any skeleton frame. It is made up of the cheapest rolled sections—that is, angles and plates,—and has many advantages for fire-proofing and building in with the masonry. Then again, the greatest amount of metal is farthest from the neutral axis. This column section has been extensively used by the P. R. R. Co. in all their outside work. For strength and accessibility for painting, it seems to have no superior.

FIG. 22.

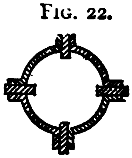


FIG. 23.

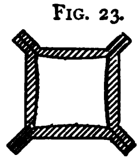


FIG. 24.

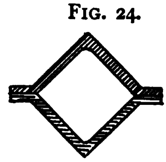
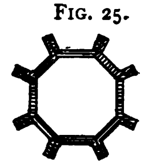


FIG. 25.



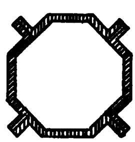


FIG. 26.

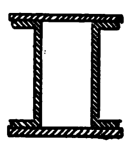


FIG. 27.

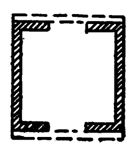


FIG. 28.

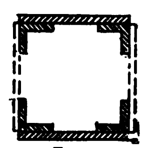


FIG. 29.

There is another shape which is more or less used, such as that shown at Fig. 22, the Phœnix column, and other new commercial shapes, such as could be conveniently rolled, as Figs. 23, 24, 25, and 26.

Fig. 27 may be either of plates and channels or latticed channels, as in Fig. 28.

In Fig. 29 angles are used at each corner; two sides may be latticed and the other sides solid plates, or all sides may be latticed.

Any of these sections, if used in the ordinary buildings, will carry the load usually placed upon them, with the unit strain required by the building laws, providing the columns are well made, with the loads symmetrically applied, as is usual in the side walls of the skeleton frame, where three of the four sides of the columns connect with the structure. Almost all of the above sections will be fully explained in detail in the examples of actually constructed buildings in the following pages, and the preference can be given to that shape which will best serve the purpose of the building to be erected.

The Advantages and Disadvantages of Different Shapes of Compound Sections.—There are many points of advantage and disadvantage to each shape which must be carefully considered before deciding the proper column to use—*i.e*:

1. **Cost.**—The cost of the column when in its finished state, which means the shapes used in the section and the number of holes to be punched and riveted. In the Z-bar column, Fig. 14, there are five different members, but only two lines of rivet-holes. Fig. 18 of plates and angles has the same number of members and rivet-holes. Z-bars and angles are probably as easy sections to roll as any commercial shapes. These columns have the advantage in only requiring two lines of rivet-holes to be punched and riveted. But if a heavier section is required, plates are placed on the outside, then six lines of holes and rivets are required. Then they become more expensive, and in Fig. 15 all the material inside the square is theoretically lost.

The column section, Fig. 19, is used in some of our noted buildings, and when not exceeding six lines of rivets is as satisfactory as Z-bar columns.

In Fig. 21 the same simple shapes are used as in Fig. 20, and is to be preferred in cost to Fig. 17.

Figs. 17 and 20 have eight lines of rivets. Fig. 21 has ten lines. The Phœnix column, Fig. 22, is a patented shape, and

only rolled by one rolling mill, but it has its advantages in having only four lines of rivets in its smallest section; but the area can be increased by simply placing fillers between the segments, and it still only requires four lines. They are also made in eight sections.

The same remarks might apply to the sections in Figs. 23, 24, 25, and 26, although the author is not aware that Figs. 23 and 24 are in the market.

Figs. 27, 28, and 29 are made up of ordinary commercial shapes that can be bought at any rolling mill. These sections require four and eight lines of rivets.

2. Availability of Material.—It is best to make up the column sections of shapes manufactured by all the rolling mills, and not those patented and only available from special places.

Z-bars are at the present manufactured generally throughout the country, and all rolling mills make angles, plates, channels, and beams.

Fig. 22 is only manufactured by the Phœnix Iron Works, of Phœnixville, Pa.; Fig. 26 by Carnegie, Phipps & Co., of Pittsburg, Pa.

3. The Advantages of Different Shape Columns for Connections.—It is an important question, and no doubt a serious one, to select the best column which will make the strongest and stiffest connections with the wall and floor girders. When there is only one beam or girder at the same level and on opposite sides, a satisfactory detail can be made for almost any of the above sections; but when arrangements of the beams and girders are irregular, both as to position and to height, those columns which present the plainest and least irregular surfaces are the ones which should be selected. By glancing at the details of column connections and the various examples, the force of the above remarks will become evident.

Z-bar and plate columns, as Figs. 14 to 21, are the best—depending, of course, upon the size of columns and size of girders forming the connection.

The Phœnix columns presented very many difficulties in their early use; but those points seem to have been remedied, so that the form of connections are very much improved.

By means of cross-pintle connections, as explained further under column connections, it is possible to make a continuous column from the basement to the roof, in which the joints are stronger laterally than the body of the column, and the connections are practically 25% less in weight than the usual form of plate and brackets.

The New York Building Law Relating to the Strength of Columns.—"The strength of all columns and posts shall be computed according to Gordon's formula,* and the crushing-weights in pounds per square inch of section, for the following-named materials shall be taken as coefficients in said formula, viz.—*Cast iron*, 80,000; *wrought or rolled iron*, 40,000; *rolled steel*, 40,000. The factors of safety shall be as 1 to 4 for all posts, columns, and other vertical supports when of wrought iron or rolled steel.

---

* Gordon's formula for the ultimate strength of columns:

$$\text{Fixed ends.}\quad \frac{P}{A} = \frac{S}{1 + K\frac{l^2}{d^2}}.$$

A = area, d = least side in inches, l = length in feet, P = load, S = total compression; unit stress, 80,000 lbs. for cast-iron, 40,000 for rolled-iron or steel.

$K = \frac{1}{800}$ for cast-iron; $K = \frac{1}{3000}$ wrought-iron or steel.

The quantity K cannot be determined theoretically; its value varies with the form of cross-section as well as with the kind of material and the arrangement of the ends of the columns.

The values of S are in pounds per square inch, while those of K are in abstract numbers.

All cast-iron, wrought iron, or rolled steel columns shall be made true and smooth at both ends, and shall rest on iron or steel bed-plates, and have iron or steel cap-plates, which shall also be made true.

In all buildings hereafter erected or altered, where any iron or steel column or columns are used to support a wall or part thereof, excepting a wall fronting on a street, and columns located below the level of the sidewalk which are used to support exterior walls or arches over vaults, the said column or columns shall be constructed double—that is, an outer and inner column. The inner column alone to be of sufficient strength to sustain safely the weight to be imposed thereon, or such other iron or steel columns of sufficient strength and so constructed as to secure resistance to fire, may be used as may be approved by the superintendent of buildings.

Cast-iron posts or columns which are to be used for the support of wooden or iron girders or brick walls, not cast with one open side or back, before being set in place, shall have a $\frac{3}{8}$-inch hole drilled in the shaft of each post or column by the manufacturer or contractor furnishing the same, to exhibit the thickness of the castings; and any other similar-sized hole or holes the superintendent of buildings or his duly authorized representatives may require shall be drilled in the said posts or columns by the said manufacturer or contractor at his own expense.

Iron posts or columns cast with one or more open sides and backs shall have solid iron plates on top of each, to prevent the passage of smoke or fire through them from one story to another, excepting where pierced for the passage of pipes.

No cast-iron post or column shall be used in any building of a less average thickness of shaft than $\frac{3}{4}$ of an inch; nor shall it have an unsupported length of more than 20 times its least lateral dimensions or diameter.

No wrought-iron or rolled-steel column shall have an unsup-

ported length of more than 30 times its least lateral dimensions or diameter; nor shall its metal be less than $\frac{1}{4}$ of an inch in thickness.

All cast-iron, wrought-iron, and steel columns shall have their bearings faced smooth and at right angles to the axis of the column, and when one column rests upon another column they shall be securely bolted together.

Strength of Cast-iron Columns.—We owe our knowledge of the *strength of cast-iron columns* chiefly to the experiments of Mr. Eaton Hodgkinson, in the year 1840. These were very numerous and to a certain degree comprehensive, embracing over two hundred examples.

As deduced from these experiments it was found that where cylindrical cast-iron columns were shorter than thirty external diameters, the weight required to break them by bending is so great that the crushing force becomes sensible, and the column yields to the combined effect of the forces. But in a long column (where the length exceeds thirty external diameters), although the pressure contributes to break it by crushing as well as by flexure or bending, yet the column yields from bending with a weight which is insufficient to sensibly affect it by crushing alone. It was found that when the pressure on the column exceeded *one fourth* of the *breaking weight*, a change or derangement of the metal took place. Therefore *one fifth* the crushing weight is as great a pressure as can be put upon cast-iron columns without having their *ultimate* strength decreased by incipient crushing; provided the thickness of metal in column is uniform, with turned ends, secured top and bottom and bolted through flanges.

If the column is secured by an uncertain method, it is safer to use *one sixth* the crushing weight.

It is obvious, therefore, that it will not do to take the table on page 30 as a guide, unless the columns are of uniform thickness throughout, of good metal, with cores made in one piece,

castings reasonably perfect and straight, the ends turned off true in a lath in planes at right angles with their axis, and set up perpendicularly in the building.

Mr. Hodgkinson, in his experiments, found that columns with rounded ends can sustain only about *one third* the weight of those with flat ends carefully fitted, with the ends at right angles to the axis of the column. In the ordinary mode of chipping off (cutting with a chisel) the ends of a column in an unfinished state, the inequalities of the bearing surfaces cause the weight to rest on a few points on the ends, and it is almost impossible that the ends shall be at right angles with the axis. The safe weight a column can sustain in such cases is considered to be about *two thirds* of one turned true.

A few experiments were also made on columns with rounded ends, and other forms than cylindrical. Square columns had an average breaking weight about 58 per cent greater than cylindrical columns of diameters equal to the sides of the squares. A pillar of the section +, 90.75 inches long, 3 inches across, and the ribs 0.48 inch thick, had a breaking weight 63 per cent greater than the computed breaking weight of a solid cylindrical column of the same weight and length. A hollow cylindrical column of the same weight and length, and of an external diameter equal to the width of the +, has a computed breaking weight about double that found by experiment for the form +.

A pillar of the section **H**, 3 inches in height and 2.5 inches in width, of the same length and nearly the same sectional area as the preceding, had a breaking weight about 2.6 times the computed breaking weight of a solid cylindrical pillar of the same weight and length.

A hollow pillar, 3 inches in external diameter and of the same weight and length, has a computed breaking weight about 19 per cent greater than was found by experiment for the **H**-section. The **H**-section being built solidly in the brick

work, the result would no doubt be quite different from Mr. Hodgkinson's tests. The results would be nearly those of the square and circular columns.

Table Giving the Strength of Hollow Cast Columns.— In computing the weight to be sustained by a column, it is not sufficient to consider only the weight appropriate to that particular use for which it is intended; but the weight should be estimated for any use to which the building may be applied, with full allowance for floors and the weights to be placed thereon. It is not safe to take the average weight sustained on each column, as some columns will have more or less on them than the average, and will be loaded more on one side than the other; besides, they are subject to concussions from bodies falling on a floor above, or may receive a lateral blow from goods falling against them in transmission.

Great allowance should also be made for columns that are subject to vibrations caused by machinery, etc.

The following table gives the *ultimate* strength of round and square cast-iron columns, in pounds per square inch of sectional area. The numbers in column $\frac{l}{d}$ = the length divided by the *least* diameter each taken in inches.

$\frac{l}{d}$	Round.	Square.	$\frac{l}{d}$	Round.	Square.
5	75,300	76,200	17	46,444	50,700
6	73,400	74,630	18	44,200	48,540
7	71,270	72,860	19	42,100	46,460
8	68,970	70,920	20	40,000	44,450
9	66,530	68,850	21	38,100	42,510
10	64,000	66,670	22	36,200	40,650
11	61,420	64,410	23	34,460	38,870
12	58,820	62,110	24	32,790	37,175
13	56,240	59,890	25	31,220	35,560
14	53,860	57,470	26	29,740	34,010
15	51,200	55,170	27	28,340	32,550
16	48,780	52,910	28	27,030	31,150

Factors of Safety for Cast-iron Columns.

(*a*) If column is accurately turned to a true plane and its bearing surfaces are perfectly true, take *one fifth* of ultimate strength.

(*b*) If column has turned ends and is set with the usual care, as in ordinary buildings, take *one sixth* of ultimate strength.

(*c*) If the ordinary mode of chipping off ends as with a chisel is employed, take *one eighth* of ultimate strength.

EXAMPLE 1. What safe load will a 12-inch-diameter column 1 inch thick, 15 feet long, support with a safety factor of 5, or one fifth the ultimate strength?

$$\frac{l}{d} = \frac{180}{12} = 15.$$

Opposite this number for round columns is 51,200 pounds, and dividing this by 5 we get 10,240 pounds, safe load per square inch of sectional area.

A 12″ dia. area = 113.10 sq. in.
" 10″ " " = 78.54 " "

34.56 = area of a 12″ dia. column 1″ thick.

Then 34.56 inches × 10.240 = 353,894 pounds or 177 tons, total safe load the column will support.

EXAMPLE 2. What safe load will a 10-inch square column 1 inch thick, 10 feet long, support with a safety factor of 6, or one sixth the ultimate strength?

$$\frac{l}{d} = \frac{120}{10} = 12.$$

Opposite this number for square columns is 62,110, which

divided by 6 gives 10,352 pounds, safe load per square inch of sectional area.

Area of column = 36 inches × 10,352 = 372,672 pounds or 186 tons, the total safe load the column will support.

Strength of Wrought-iron and Steel Columns.— Wrought-iron and steel columns fail either by deflecting bodily out of a straight line, or by the buckling of the metal between rivets or other points of support.

Both actions may take place at the same time, but if the latter occurs alone it may be an indication that the rivet spacing or the thickness of the metal is insufficient.

Until a few years ago we have had no experimental knowledge on this subject beyond the experiments of Hodgkinson, which have furnished the constants for Hodgkinson's and also for Gordon's formula.

Then we had Euler's formula, where it is assumed that for any given material there is a certain definite ratio of length to diameter below which a column will give away by direct crushing, while one whose ratio of length to diameter is greater will give way *wholly* by transverse strain.

Hodgkinson's empirical formulæ were based upon his experiments upon small columns of a variety of ratios of length to diameter.

Then Gordon's formula, where it is assumed that all columns give way by a combination of *crushing* and *bending*.

The formula which seems to most satisfactorily represent the result of experiments is that of Gordon, or, as it is sometimes referred to, "Gordon's formula modified by Rankine;" but the best usage gives to it the name of Rankine's formula. The disagreement of the formulæ already referred to has led to the proposal of a number of similar formulæ, each having its constants determined to suit certain definite set of tests, and all these thus proposed must be classed as empirical formulæ, and applied within the cases experimented upon.

In 1881, Mr. Clark, of the firm of Clark, Reeves & Co., presented to the American Society of Civil Engineers a report of a number of tests on full-sized Phœnix columns, made for them at the Watertown Arsenal, together with a comparison of the actual breaking weights with those which would have been obtained by using the common form of Gordon's formula for *wrought iron:*

$$\frac{P}{A} = \frac{36000}{1 + \frac{l^2}{36000r^2}};$$

where P = breaking weight in pounds, A = area of section in square inches, l = length in inches, r = least radius of gyration in inches. The table is as follows:

No. of Experiment.	Length of Column. Ft.	Ratio of Diameter to Length.	Weight. Lbs.	Sectional Area. Sq. In.	Total Compression under Loads.		Elastic Limit.		Ultimate Strength.		Total Ultimate Strength. in lbs., by Gordon's Formula.
					Lbs. 200,000.	Lbs. 300,000.	Total lbs.	Lbs. per sq. in.	Total lbs.	Lbs. per sq. in.	
1	28	42	1,142	12.062	0.190	.....	....	.....	424,000	35,150	330,146
2	28	42	1,153	12.181	0.186	....	....	... ..	416,000	34,150	333,459
3	25	37½	1,034	12.233	.....	0.255	342,000	27,960	431,500	35,270	352,013
4	25	37½	1,023	12.100	0.168	0.264	....	......	424,000	35,040	348,119
5	22	33	920	12.371	0.160	0.243	....	......	440,000	35,570	372,837
6	22	33	......	12.311	0.152	0.236	....	......	423,000	34,360	371,043
7	19	28½	773	12.023	.....	0.198	....	....	425,200	35,365	377,955
8	19		777	12.087	0.139	0.213	354,000	29,290	446,000	36,900	380,197
9	16	24	650	12.000	0.120	.....	....	......	439,000	36,580	391,701
10	16		650	12.000	0.116	.....	....	......	439,000	36,580	391,701
11	13	19½	536	12.185	0.092	0.142	342,000	28,890	449,000	36,857	410,660
12	13		531	12.009	0.091	.....	....	......	449,000	37,200	406,886
13	10	15	415	12.248	.....	0.110	330,000	26,940	446,800	36,480	423,886
14	10		418	12.339	.....	0.109	350,000	28,360	449,100	36,397	427,047
15	7	10½	291	12.265	0.054	.....	360,000	29,350	468,000	38,157	433.021
16	7		284	11.962	.....	.....	354,000	29,590	517,000	43,300	469,324
17	4	6	164	12.081	0.031	. ...	....	......	598,000	49,500	432,132
18	4	6	164½	12.119	0.025	0.042	340,000	28,050	621,000	51,240	433,507

Other tests made at the Watertown Arsenal will next be given.

WROUGHT-IRON COLUMNS.

LATTICED COLUMN—CHANNEL BARS SPACED 8″ APART.

	Size of Bars.	Length.		Sectional Area.	Lattice Spacing.	Ultimate Strength.		Manner of Failure.
						Actual.	Per sq. in.	
	in.	ft.	in.	sq. in.	in.	lbs.	lbs.	
Flat ends	6	10	0	4.760	18	174,800	36,720	Channels buckled.
" "	6	10	0	4.670	18	165,000	35,330	" "
Pin ends	6	12	0	4.600	18	159,800	34,740	Horizontal deflection.
" "	6	15	0	4.480	18	151,500	33,820	" "
" "	6	17	6	4.660	18	152,600	32,750	" "
" "	6	20	6	4.660	18	136,000	29,180	" "
" "	6	22	6	4.570	18	139,800	30,590	" "
" "	6	25	0	4.710	18	110,000	23,350	" "
" "	6	27	6	4.690	18	102,500	21,850	" "
" "	6	30	0	4.700	18	69,300	14,740	" "
" "	8	13	4	7.520	18	261,800	34,810	Defl. upward; ch. bars buckled.
" "	8	16	8	7.480	18	254,100	33,970	" horizon. " "
" "	8	20	0	7.550	18	246,200	32,610	" " " "
" "	8	23	4	7.990	18	257,500	32,230	" "
" "	8	26	8	7.780	18	243,900	31,350	" "
" "	8	30	0	7.810	18	194,100	24,850	" "
" "	10	12	6	9.680	22	344,120	35,550	Channel bars buckled.
" "	10	16	8	9.550	22	323,200	33,840	" " "
" "	10	20	10	9.740	22	330,000	33,880	" " "
" "	10	25	0	10.040	22	342,700	34,130	" " "
" "	10	29	2	9.300	22	299,300	32,180	Deflection horizontally.
" "	12	20	0	11.980	22	411,600	34,360	Channel bars buckled.
" "	12	25	0	12.144	22	400,000	32,940	" " "
" "	12	25	0	11.910	22	407,800	34,240	" " "
" "	12	30	0	12.180	22	385,000	31,610	" " "
" "	12	30	0	12.540	22	393,000	31,340	Deflection horizontally.

TESTS OF Z-BAR COLUMNS.

Some tests were made on *iron* Z-bar column by C. L. Strobel, C.E., and reported in the Trans. Am. Soc., C.E. Paper, April, 1888. These tests were fifteen full-sized specimens, in which the central web-plates were replaced by lattice bars. The results for lengths ranging from 64 to 88 radii showed an average ultimate resistance *per square inch* of 35,650 lbs.

The tabulated values are based upon the formula,

$$46{,}000 - 125\frac{l}{r},$$

for lengths exceeding 90 radii and 35,000 for lengths equal to or less than 90 radii.

Section of column: 4 Z-bars, $2\frac{1}{2}'' \times 3'' \times 2\frac{1}{2}''$—(latticed).
Radius of gyration (latticed bars not considered) = 2.05″.

Length of Column.	Sectional Area. Square Inches.	Ultimate Strength by Actual Tests. Lbs. per Square Inch.	Ratio of Length to Least Radius of Gyration.	Ultimate Strength by Formula $46,000 - 125\frac{l}{r}$.
15′-0″	9.480	34,600	88	35,000
15′-0″	9.280	36,600	88	35,000
19′-0⅝″	9.241	33,800	112	32,200
19′-0⅝″	10.104	33,700	112	32,200
22′-0″	9.286	30,700	129	29,900
22′-0″	9.286	29,500	129	29,900
22′-0″	9.286	30,700	129	29,000
25′-0″	9.156	28,100	146	27,750
25′-0″	9.456	28,000	146	27,750
25′-0″	9.516	28,400	146	27,750
28′-0″	9.375	27,700	164	25,500
28′-0″	9.643	28,000	164	25,500
28′-0″	9.375	27,600	164	25,500

WROUGHT-IRON BOX COLUMNS WITH FLAT ENDS.

Style of Column.	Total Length.	Sectional Area.	Ultimate Strength: Total Lbs.	Ultimate Strength: Pounds per Sq. Inch.	Manner of Failure.
Two 6″ channels 5.5 inches apart, flanges turned out with two ¼-inch cover-plates	10 7.9	12.08	383,200	31,722	Plates buckeled between the rivets.
do. do.	10 7.9	11.11	372,900	33,564	
Two 8″ channels 7.6 inches apart, flanges turned out with two $\frac{5}{16}$-inch cover-plates	13 11.8	17.01	594,500	34,950	do.
do. do.	13 11.8	17.80	633,600	35,595	Triple flexure.
Four plates connected with four angles forming a box 7″ × 7½″ inside	13 11.9	15.74	517,000	32,846	Buckling plates.
Plates and angles all $\frac{5}{16}$″ thick	13 11.6	15.84	555,200	35,050	Buckling plates.
do. do.	20 7.63	15.68	517,500	33,003	Deflecting upward.
do. do.	20 7.80	15.56	536,900	34,505	Buckling plates.
Single web columns with 3½-inch pin-ends.					
One $\frac{5}{16}$″ web 8″ wide with four angles, and 8″ channels used in place of cover-plates, flanges outward	13 4	15.34	47,500	30,965	Deflecting upward in plane of pin.

Strength of Steel Columns.—Experiments thus far upon steel struts indicate that for lengths up to 90 radii of gyration their ultimate strength is about 20 per cent. higher than for wrought-iron. Beyond this point the excess of strength diminishes until it becomes zero at about 200 radii. After passing this limit the compression resistance of steel and wrought-iron seems to become practically equal.

ULTIMATE STRENGTH OF WROUGHT-IRON COLUMNS.

Square ends. By formula $\dfrac{40000}{1 + \dfrac{(12l)^2}{36000r^2}}$,

l = length in feet, r = least radius of gyration in inches.

To be used for columns not cylindrical. For safe load take $\frac{1}{4}$ the ultimate.

$\frac{l}{r}$	Ultimate Strength in lbs. per sq. in.	$\frac{l}{r}$	Ultimate Strength in lbs. per sq. in.	$\frac{l}{r}$	Ultimate Strength in lbs. per sq. in.	$\frac{l}{r}$	Ultimate Strength in lbs. per sq. in.	$\frac{l}{r}$	Ultimate Strength in lbs. per sq. in.
3.0	38,610	6.0	34,970	9.0	30,210	12.0	25,380	15.5	20,290
3.2	38,430	6.2	34,670	9.2	29,880	12.2	25,070	15.8	20,020
3.4	38,230	6.4	34,370	9.4	29,550	12.4	24,770	16.0	19,760
3.6	38,030	6.6	34,060	9.6	29,230	12.6	24,470	16.2	19,510
3.8	37,820	6.8	33,750	9.8	28,900	12.8	24,170	16.5	19,150
4.0	37,590	7.0	33,440	10.0	28,570	13.0	23,870	16.8	18,790
4.2	37,360	7.2	33,130	10.2	28,250	13.2	23,570	17.0	18,550
4.4	37,120	7.4	32,810	10.4	27,920	13.5	23,140	17.2	18,320
4.6	36,870	7.6	32,490	10.6	27,600	13.8	22,700	17.5	17,980
4.8	36,620	7.8	32,170	10.8	27,270	14.0	22,420	17.8	17,640
5.0	36,360	8.0	31,850	11.0	26,950	14.2	22,150	18.0	17,420
5.2	36,090	8.2	31,520	11.2	26,640	14.5	21,740	18.2	17,200
5.4	35,820	8.4	31,190	11.4	26,320	14.8	31,320	18.5	16,880
5.6	35,540	8.6	30,870	11.6	26,000	15.0	21,050	18.8	16,570
5.8	35,260	8.8	30,540	11.8	25,690	15.2	20,790	19.0	16,370

Radius of Gyration.—In order to find the strength of long columns we need to know r^2, or the square of the radius of gyration.

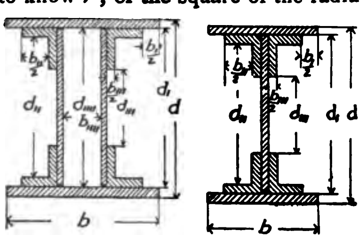


FIG. 30.

We have, in general,

$$r^2 = \frac{I}{A}, \quad \text{or} \quad r = \sqrt{\frac{I}{A}},$$

where r is the radius of gyration, I is the moment of inertia of the cross-section to the required axis, and A is the area of the cross-section.

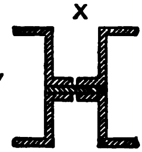
The moment of inertia of such built-up sections, as in Fig. 30, with reference to the axis through its own centre of gravity parallel to its breadth, is $\frac{1}{12}bd^3$, or

$$I = \frac{bd^3 - (b_{,}d_{,}^3 + b_{,,}d_{,,}^3 + b_{,,,}d_{,,,}^3 + b_{,,,,}d_{,,,,}^3)}{12}$$

N.B.—If $\frac{b_{,}}{2} = \frac{3}{4}$ of an inch, $b_{,} = 1\frac{1}{2}$ inches.

ELEMENTS OF Z-BAR COLUMNS.

I = Moment of inertia.
A = Area.

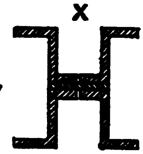


R = Radius of gyration.

THE THICKNESS OF WEB PLATE AND Z-BAR IS THE SAME.

Size of Z-Bar in Inches.	7″ Web Plate. 7¼″ Face to Face.					7¼″ Web Plate. 7½″ Face to Face.				
	Area of 4 Z-Bars and 1 Plate.	Axis XX.		Axis YY.		Area of 4 Z-Bars and 1 Plate	Axis XX.		Axis YY.	
		I.	R^2.	I.	R^2.		I.	R^2.	I.	R^2.
3½ × 6 × 3½ × ⅜	20.99	264.18	12.59	287.91	13.72	21.17	299.34	14.14	287.91	13.60
3 9/16 × 6 1/16 × 3 9/16 × 7/16	24.62	306.41	12.45	346.95	14 09	24.84	347.30	13.98	346.95	13.97
3⅝ × 6⅛ × 3⅝ × ½	28.26	347.81	12.31	409.27	14.48	28.51	392.86	13.78	409.28	14.36
3½ × 6 × 3½ × 9/16	30.66	365.24	11.91	426.30	13.90	30.94	415.23	13.42	426.31	13.78
3 9/16 × 6 1/16 × 3 9/16 × ⅝	34.22	403.02	11.78	489.32	14.30	34.53	458.45	13.28	489.33	14.17
3⅝ × 6⅛ × 3⅝ × 11/16	37.81	440.25	11.64	555.79	14.70	38.16	500.93	13.13	455.80	14.57
3½ × 6 × 3½ × ¾	39.81	448.24	11.26	562.41	14 13	40.19	511.45	12.73	562.42	13.99
3 9/16 × 6 1/16 × 3 9/16 × 13/16	43.21	481.06	11.13	628.31	14.54	43.61	549.08	12.59	628.33	14.41
3⅝ × 6⅛ × 3⅝ × ⅞	46.77	514.73	11.00	699 07	14.95	47.20	587.80	12.45	699.10	14.81
	6¼″ Web Plate. 6½″ Face to Face.					7″ Web Plate. 7¼″ Face to Face.				
3 3/16 × 5 × 3 3/16 × 5/16	15.47	169.65	10.97	147.39	9.53	15.63	193.91	12.41	147.39	9.43
3¼ × 5 1/16 × 3¼ × ⅜	18.64	202.04	10.84	183.47	9.84	18.83	231.00	12.27	183.47	9.74
3 5/16 × 5⅛ × 3 5/16 × 7/16	21.84	233.93	10.71	223.00	10.21	22.06	267.61	12.13	223.00	10.11
3 7/32 × 5 × 3 7/32 × ½	24.17	249.97	10.34	234.39	9.70	24.42	287.67	11.78	234.39	9.60
3 9/32 × 5 1/16 × 3 9/32 × 9/16	27.30	279.93	10.25	273.72	10.03	27.58	321.22	11.65	273.72	9.93
3 11/32 × 5⅛ × 3 11/32 × ⅝	30.46	308.80	10.14	315.55	10.36	30.78	354.42	11.52	315.56	10.25
3¼ × 5 × 3¼ × 11/16	32.31	316.97	9.81	320.08	9.91	32.65	364 83	11.17	320.09	9.80
3 5/16 × 5 1/16 × 3 5/16 × ¾	35.44	343.48	9.69	362 93	10.24	35.81	395.52	11.04	362.95	10.14
	6″ Web Plate. 6¼″ Face to Face.					6¼″ Web Plate. 6½″ Face to Face.				
2⅞ × 4 × 2⅞ × ¼	10.78	101.90	9.45	65.72	6.10	10.91	117.62	10 78	65.72	6.02
2 15/16 × 4 1/16 × 2 15/16 × 5/16	13.52	126.20	9.34	85.86	6.35	13.67	145.72	10.66	85.86	6.28
3 × 4⅛ × 3 × ⅜	16.25	149.91	9.23	107.47	6.61	16.44	173.18	10.53	107.47	6.54
2 31/32 × 4 × 2 31/32 × 7/16	18.47	166.01	8.99	115.63	6.26	18.68	192.14	10.29	115.64	6.19
3 1/32 × 4 1/16 × 3 1/32 × ½	21.24	188.60	8.88	138.44	6.52	21.49	218.39	10.16	138.45	6.44
3 3/32 × 4⅛ × 3 3/32 × 9/16	24.02	210.67	8.77	163.09	6.79	24.30	244.05	10.04	163.10	6.71
3 1/16 × 4 × 3 1/16 × ⅝	25.87	221.21	8.55	166.90	6.45	26.18	256.76	9.83	166.91	6.39
3⅛ × 4 1/16 × 3⅛ × 11/16	28.69	242.12	8.44	192.70	6 72	29.03	281.15	9.69	192.70	6.64
3 3/16 × 4⅛ × 3 3/16 × ¾	31.50	262.65	8.32	220.68	7.01	31.88	305.12	9.57	220.70	6.92
	5¼″ Web Plate. 5½″ Face to Face.					6″ Web Plate. 6¼″ Face to Face.				
2 11/16 × 3 × 2 11/16 × ¼	9.14	72.59	7.94	31.74	3.47	9.26	84.82	9.16	31 74	3.43
2 11/16 × 3 1/16 × 2 11/16 × 5/16	11.48	90.17	7.85	42.14	3.67	11.64	105.31	9.05	42.15	3.62
2¾ × 3⅛ × 2¾ × ⅜	13.82	107.05	7.75	53.40	3.86	14 01	125.14	8.93	53.41	3.81
2 21/32 × 3 × 2 21/32 × 7/16	15.53	115.58	7.44	55.61	3.58	15.75	135.63	8.61	55.61	3.53
2 23/32 × 3 1/16 × 2 23/32 × ½	17.75	130.45	7.35	67.20	3.79	18.00	153.14	8.51	67.20	3.73

ELEMENTS OF Z-BAR COLUMNS.



I = Moment of inertia. r = Radius of gyration.

THE THICKNESS OF WEB PLATE AND Z-BAR IS THE SAME.

Size of Z-Bar in Inches.	8″ Web Plate. 8¼″ Face to Face.					8¼″ Web Plate. 8½″ Face to Face.				
	Area of 4 Z-Bars and 1 Plate.	Axis XX.		Axis YY.		Area of 4 Z-Bars and 1 Plate.	Axis XX.		Axis YY.	
		I.	R^2.	I.	R^2.		I.	R^2.	I.	R^2.
3½ × 6 × 3½ × ⅜	21.36	337.17	15.78	287.92	13.48	21.55	377.65	17.52	287.92	13.36
3 9/16 × 6 1/16 × 3 9/16 × 7/16	25.06	391.37	15.62	346.96	13.85	25.28	438 55	17.35	346.96	13.73
3⅝ × 6⅛ × 3⅝ × ½	28.76	444.57	15.46	409.28	14.23	29.01	498.35	17.18	409 29	14.11
3½ × 6 × 3½ × 9/16	31.22	469.16	15.03	426.32	13.65	31 50	527.03	16.73	426.33	13 53
3 9/16 × 6 1/16 × 3 9/16 × ⅝	34.84	518.19	14.88	489.34	14.05	35.15	582.27	16.65	489.35	13.92
3⅝ × 6⅛ × 3⅝ × 11/16	38.50	566.43	14.72	555.82	14.44	38.84	636.74	16.39	555.83	14.31
3½ × 6 × 3½ × ¾	40 56	579.76	14.29	562.44	13.87	40.94	653.06	15.95	562.46	13.74
3 9/16 × 6 1/16 × 3 9/16 × 13/16	44.02	622.59	14.14	628.36	14.27	44.43	701.62	15.79	628.38	14.14
3⅝ × 6⅛ × 3⅝ × ⅞	47.64	666.83	14.00	699.13	14.67	48.08	751.66	15.63	699.15	14.54
	7½″ Web Plate. 7¾″ Face to Face.					8″ Web Plate. 8¼″ Face to Face.				
3 3/16 × 5 × 3 3/16 × 5/16	15.78	220.13	13.95	147.39	9.35	15.94	248.29	15.58	147.39	9.25
3¼ × 5 1/16 × 3¼ × ⅜	19.01	262.32	13.80	183.47	9.65	19.20	296.02	15.42	183.48	9.56
3 5/16 × 5⅛ × 3 5/16 × 7/16	22.28	303.96	13.64	223.00	10.01	22.50	343.21	15.25	223.01	9.91
3 7/32 × 5 × 3 7/32 × ½	24.67	327.56	13.28	234 40	9.50	24.92	370.53	14.87	234.40	9.41
3 9/32 × 5 1/16 × 3 9/32 × 9/16	27.96	365.87	13.13	273.73	9.83	28.14	414.08	14.72	273.74	9.73
3 11/32 × 5⅛ × 3 11/32 × ⅝	31.09	403.93	22.99	315.57	10.15	31.40	457.31	14.56	315.58	10.05
3¼ × 5 × 3¼ × 11/16	33.00	416.75	12.63	320.10	9.70	33.34	472.79	14.18	320.12	9.60
3 5/16 × 5 1/16 × 3 5/16 × ¾	36.19	452.01	12.49	362.96	10.03	36.56	513.78	14.05	362.98	9.93
	7″ Web Plate. 7¼″ Face to Face.					7¼″ Web Plate. 7½″ Face to Face.				
2⅞ × 4 × 2⅞ × ¼	11.03	134.71	12.21	65.72	5.96	11.16	153.17	13.72	65.72	5.89
2 15/16 × 4 1/16 × 2 15/16 × 5/16	13.83	166.97	12.07	85.86	6.21	13.98	189.95	13.59	85.86	6.14
3 × 4⅛ × 3 × ⅜	16.63	198.52	11.94	107.47	6.46	16.81	225.94	13.44	107.47	6.39
2 31/32 × 4 × 2 31/32 × 7/16	18.90	220.75	11.68	115.64	6.12	19.12	251.40	13.15	115.64	6.05
3 1/32 × 4 1/16 × 3 1/32 × ½	21.74	250.90	11.54	138.45	6.37	21.99	286.10	13.01	138.46	6.30
3 3/32 × 4⅛ × 3 3/32 × 9/16	24.58	280.48	11.41	163.10	6.64	24.86	319.96	12.87	163.11	6.56
3 1/16 × 4 × 3 1/16 × ⅝	26.50	295.54	11.15	166.92	6.30	26.81	337.59	12.59	166.93	6.23
3⅛ × 4 1/16 × 3⅛ × 11/16	29.37	323.83	11.03	192.73	6 56	29.72	370.17	14.45	192.74	6.49
3 3/16 × 4⅛ × 3 3/16 × ¾	32.25	351.60	10.90	220.72	6.84	32.63	402.09	12.32	220.73	6.77
	6¾″ Web Plate. 6¾″ Face to Face.					7″ Web Plate. 7¼″ Face to Face.				
2⅝ × 3 × 2⅝ × ¼	9.39	98.12	10.45	31.74	3.38	9.51	112 65	11.85	31.74	3.34
2 11/16 × 3 1/16 × 2 11/16 × 5/16	11.79	121.99	10.35	42.15	3.58	11.95	140.07	11.71	42.15	3.53
2¾ × 3⅛ × 2¾ × ⅜	14.20	144.98	10.21	53.41	3.76	14.39	166.60	11.58	53.41	3.71
2 21/32 × 3 × 2 21/32 × 7/16	15.96	157.65	9.88	55.62	3.49	16.18	181.67	11.23	55.62	3.44
2 23/32 × 3 1/16 × 2 23/32 × ½	18.25	178.09	9.76	67.21	3.68	18.50	205.32	11.10	67.21	3.63

SAFE LOADS IN TONS OF 2000 LBS.

* Steel Z-Bar Columns, Square Ends.

Allowed strains per square inch for steel, safety factor 4:

12,000 lbs. for lengths of 90 radii or under.

$17,100-57\frac{l}{r}$ for lengths over 90 radii.

6″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 3″ deep and 1 web plate 5¼″ × thickness of Z-bars.

Length of Column in Feet.	¼ Metal = 31.7 lbs. = 9.31 sq. in. r (min.) = 1.86.	5/16 Metal = 39.8 lbs. = 11.7 sq. in. r (min.) = 1.90.	⅜ Metal = 46.2 lbs. = 13.6 sq. in. r (min.) = 1.88.	7/16 Metal = 54.3 lbs. = 16.0 sq. in. r (min.) = 1.93.	½ Metal = 59.9 lbs. = 17.6 sq. in. r (min.) = 1.90.	9/16 Metal = 67.9 lbs. = 20.0 sq. in. r (min.) = 1.95.
12 and under	55.9	70.3	81.6	95.8	105.7	119.8
14	55.7	70.3	81.6	95.8	105.7	119.8
16	52.3	66.5	76.6	91.3	99.9	114.8
18	48.8	62.3	71.7	85.6	93.6	107.8
20	45.4	58.1	66.7	79.9	87.2	100.8
22	42.0	53.9	61.8	74.3	80.9	93.8
24	38.6	49.7	56.9	68.6	74.6	86.8
26	35.2	45.5	51.9	63.0	68.2	79.8
28	31.7	41.3	47.0	57.3	61.9	72.8
30	28.3	37.1	42.0	51.7	55.5	65.8

8″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 4″ deep and 1 web plate 6¼″ × thickness of Z-bars.

Length of Column in Feet.	¼ Metal = 38.3 lbs. = 11.3 sq. in. r (min.) = 2.47.	5/16 Metal = 48.1 lbs. = 14.7 sq. in. r (min.) = 2.52.	⅜ Metal = 58.0 lbs. = 17.1 sq. in. r (min.) = 2.57.	7/16 Metal = 64.7 lbs. = 19.0 sq. in. r (min.) = 2.49.	½ Metal = 73.7 lbs. = 21.9 sq. in. r (min.) = 2.55.	9/16 Metal = 84.1 lbs. = 24.8 sq. in. r (min.) = 2.60.	⅝ Metal = 89.2 lbs. = 26.3 sq. in. r (min.) = 2.52.	11/16 Metal = 98.8 lbs. = 29.0 sq. in. r (min.) = 2.58.	¾ Metal = 108.4 lbs. = 31.9 sq. in. r (min.) = 2.63.
18 and under	67.5	84.8	102.4	114.2	131.2	148.5	157.5	174.3	191.2
20	65.0	82.5	100.5	110.5	128.2	146.4	153.3	171.3	189.6
22	61.9	78.7	95.9	105.3	122.4	139.9	146.2	163.5	181.3
24	58.8	74.8	91.3	100.1	116.5	133.4	139.1	155.8	173.0
26	55.7	71.0	86.8	94.8	110.6	126.9	132.0	148.1	164.7
28	52.6	67.1	82.3	89.6	104.7	120.3	124.8	140.4	156.4
30	49.4	63.3	77.7	84.4	98.8	113.8	117.7	132.7	148.2
32	46.3	59.5	73.2	79.2	93.0	107.3	110.6	125.0	139.9
34	43.2	55.6	68.7	74.0	87.1	100.8	103.5	117.3	131.6
36	40.1	51.8	64.1	68.7	81.2	94.3	96.4	109.6	123.3
38	37.0	48.0	59.6	63.5	75.3	87.8	89.4	101.9	115.0
40	33.9	44.1	55.0	58.3	69.5	81.3	82.2	94.2	106.7

*** From Carnegie, Phipps & Co.'s Hand Books.**

SAFE LOADS IN TONS OF 2000 LBS.

Steel Z-Bar Columns, Square Ends.

Allowed strains per square inch for steel, safety factor 4:
12,000 lbs. for lengths of 90 radii or under.
$17{,}100-57\frac{l}{r}$ for lengths over 90 radii.

10″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 5″ deep and 1 web plate 7″ × thickness of Z-bars.

Length of Column in Feet.	5/16 Metal = 53.7 lbs. = 15.8 sq. in. r (min.) = 3.08.	3/8 Metal = 64.7 lbs. = 19.0 sq. in. r (min.) = 3.13.	7/16 Metal = 75.8 lbs. = 22.3 sq. in. r (min.) = 3.18.	1/2 Metal = 83.3 lbs. = 24.5 sq. in. r (min.) = 3.10.	9/16 Metal = 94.2 lbs. = 27.7 sq. in. r (min.) = 3.15.	5/8 Metal = 105.2 lbs. = 30.9 sq. in. r (min.) = 3.21.	11/16 Metal = 111.0 lbs. = 32.7 sq. in. r (min.) = 3.13.	3/4 Metal = 122.8 lbs. = 35.8 sq. in. r (min.) = 3.18.	13/16 Metal = 132.6 lbs. = 39.0 sq. in. r (min.) = 3.25.
22 and under	94.7	114.2	133.9	147.0	166.2	185.6	196.0	214.9	234.0
24	92.8	112.6	133.1	144.6	164.8	185.3	193.6	213.9	234.0
26	89.3	108.6	128.3	139.2	158.7	178.7	186.5	206.2	226.6
28	85.8	104.4	123.5	133.8	152.7	172.1	179.3	198.5	218.4
30	82.3	100.2	118.7	128.4	146.7	165.5	172.2	190.8	210.2
32	78.8	96.1	113.8	123.0	140.7	158.9	165.0	183.1	202.0
34	75.3	91.9	109.1	117.6	134.7	152.3	157.9	175.4	193.8
36	71.8	87.8	104.3	112.2	128.7	145.7	150.7	167.8	185.6
38	68.3	83.6	99.5	106.8	122.7	139.1	143.6	160.0	177.4
40	64.8	79.4	94.7	101.4	116.7	132.5	136.5	152.3	169.1
42	61.3	75.3	89.9	96.0	110.6	125.9	129.4	144.6	160.9
44	57.7	71.1	85.1	90.6	104.6	119.3	122.2	136.9	152.7
46	54.2	67.0	80.3	85.2	98.6	112.7	115.1	129.2	144.5
48	50.7	62.8	75.5	79.8	92.6	106.1	107.9	121.5	136.3
50	47.2	58.6	70.7	74.4	86.6	99.5	100.8	113.8	128.1

12″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6″ deep and 1 web plate 8″ × thickness of Z-bars.

Length of Column in Feet.	3/8 Metal = 72.7 lbs. = 21.4 sq. in. r (min.) = 3.67.	7/16 Metal = 85.2 lbs. = 25.0 sq. in. r (min.) = 3.72.	1/2 Metal = 97.8 lbs. = 28.8 sq. in. r (min.) = 3.77.	9/16 Metal = 106.2 lbs. = 31.2 sq. in. r (min.) = 3.70.	5/8 Metal = 118.5 lbs. = 34.8 sq. in. r (min.) = 3.75.	11/16 Metal = 130.9 lbs. = 38.5 sq. in. r (min.) = 3.73.	3/4 Metal = 137.8 lbs. = 40.5 sq. in. r (min.) = 3.68.	13/16 Metal = 149.9 lbs. = 44.1 sq. in. r (min.) = 3.66.	7/8 Metal = 162.1 lbs. = 47.7 sq. in. r (min.) = 3.64.
26 and under	128.3	150.3	172.6	187.3	209.1	231.0	243.0	264.5	286.1
28	127.0	149.7	172.5	186.0	208.9	230.3	240.8	261.4	282.1
30	123.0	145.1	167.6	180.2	202.5	223.3	233.2	253.2	273.2
32	119.0	140.5	162.4	174.5	196.1	216.3	225.7	245.0	264.2
34	115.1	135.9	157.2	168.7	189.8	209.2	218.2	236.7	255.2
36	111.1	131.3	152.0	162.9	183.4	202.1	210.6	228.4	246.3
38	107.1	126.7	146.8	157.1	177.0	195.1	203.1	220.2	237.3
40	103.1	122.1	141.5	151.4	170.7	188.0	195.6	211.9	228.3
42	99.1	117.5	136.3	145.5	164.4	180.9	188.0	203.7	219.4
44	95.1	112.9	131.1	139.8	158.0	173.9	180.5	195.5	210.4
46	91.2	108.3	126.2	134.0	151.6	166.8	172.9	187.2	201.4
48	87.2	103.6	120.7	128.2	145.3	159.8	165.4	179.0	192.4
50	83.2	99.1	115.5	122.4	138.9	152.7	157.9	170.7	183.5

SAFE LOADS IN TONS OF 2000 LBS.

Steel Z-Bar Columns, Square Ends.

Allowed strains per square inch for steel, safety factor 4:
12,000 lbs. for lengths of 90 radii or under.
$17,100 - 57\frac{l}{r}$ for lengths over 90 radii.

14″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × 11/16″. 1 web plate 8″ × 11/16″. 2 side plates 14″ wide.

Length of Column in Feet.	14 × ⅜ Plates = 166.6 lbs. = 49.0 sq. in. r (min.) = 3.80.	14 × 7/16 Plates = 172.6 lbs. = 50.8 sq. in. r (min.) = 3.81.	14 × ½ Plates = 178.5 lbs. = 52.5 sq. in. r (min.) = 3.82.	14 × 9/16 Plates = 184.5 lbs. = 54.3 sq. in. r (min.) = 3.82.	14 × ⅝ Plates = 190.4 lbs. = 56.0 sq. in. r (min.) = 3.83.	14 × 11/16 Plates = 196.4 lbs. = 57.8 sq. in. r (min.) = 3.84.	14 × ¾ Plates = 202.3 lbs. = 59.5 sq. in. r (min.) = 3.85.	14 × 13/16 Plates = 208.4 lbs. = 61.3 sq. in. r (min.) = 3.85.	14 × ⅞ Plates = 214.2 lbs. = 63.0 sq. in. r (min.) = 3.85.
28 and under	294.0	304.5	315.0	325.5	336.0	346.5	357.0	367.5	378.0
30	286.6	297.2	307.7	318.3	328.9	339.5	350.0	360.4	370.9
32	277.8	288.1	298.3	308.6	318.9	329.2	339.4	349.5	359.7
34	269.0	278.9	288.9	298.9	308.9	318.9	328.8	338.6	348.6
36	260.1	269.8	279.5	289.2	298.9	308.6	318.2	327.7	337.4
38	251.3	260.7	270.1	279.5	289.0	298.3	307.6	316.8	326.2
40	242.5	251.6	260.7	269.7	278.9	288.0	297.0	306.0	315.0
42	233.7	242.5	251.3	260.1	269.0	277.8	286.4	295.1	303.8
44	224.9	233.3	241.9	250.4	258.9	267.4	275.8	284.2	292.6
46	216.0	224.3	232.4	240.7	249.0	257.2	265.2	273.3	281.5
48	207.2	215.1	223.0	230.9	238.9	246.9	254.6	262.4	270.3
50	198.4	206.0	213.6	221.3	229.0	236.5	244.0	251.5	259.1

14″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6″ × ¾″. 1 web plate 8″ × ¾″. 2 side plates 14″ wide.

Length of Column in Feet.	14 × ⅜ Plates = 173.4 lbs. = 51.0 sq. in. r (min.) = 3.75.	14 × 7/16 Plates = 179.4 lbs = 52.8 sq. in. r (min.) = 3.76.	14 × ½ Plates = 185.3 lbs. = 54.5 sq. in. r (min.) = 3.77.	14 × 9/16 Plates = 191.3 lbs. = 56.3 sq. in. r (min.) = 3.78.	14 × ⅝ Plates = 197.2 lbs. = 58.0 sq. in. r (min.) = 3.79.	14 × 11/16 Plates = 203.2 lbs. = 59.8 sq. in. r (min.) = 3.80.	14 × ¾ Plates = 209.1 lbs. = 61.5 sq. in. r (min.) = 3.80.	14 × 13/16 Plates = 215.1 lbs. = 63.3 sq. in. r (min.) = 3.81.	14 × ⅞ Plates = 221.0 lbs. = 65.0 sq. in. r (min.) = 3.82.
28 and under	306.0	316.5	327.0	337.5	348.0	358.5	369.0	379.5	390.0
30	296.7	307.2	317.8	328.3	338.9	349.4	359.9	370.5	381.1
32	287.4	297.6	307.9	318.2	328.4	338.7	348.9	359.1	369.4
34	278.1	288.0	298.0	308.0	318.0	327.9	337.8	347.8	357.8
36	268.8	278.4	288.2	297.9	307.4	317.2	326.8	336.4	346.1
38	259.5	268.8	278.3	287.7	297.0	306.4	315.7	325.1	334.5
40	250.2	259.3	268.4	277.5	286.5	295.6	304.7	313.7	322.8
42	240.9	249.7	258.5	267.3	276.1	284.8	293.6	302.4	311.2
44	231.6	240.1	248.6	257.1	265.6	274.1	282.5	291.0	299.6
46	222.4	230.5	238.7	246.9	255.1	263.4	271.5	279.7	287.9
48	213.0	220.9	228.8	236.8	244.7	252.6	260.4	268.3	276.2
50	203.7	211.3	219.0	226.6	234.2	241.8	249.4	257.0	264.6

SAFE LOADS IN TONS OF 2000 LBS.

Steel Z-Bar Columns, Square Ends.

Allowed strains per square inch for steel, safety factor 4:
12,000 lbs. for lengths of 90 radii or under.

$17,100-57\frac{l}{r}$ for lengths over 90 radii.

14″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6 1/16″ × 11/16″. 1 web plate 8″ × 11/16″. 2 side plates 14″ wide.

Length of Column in Feet.	14 × 3/8 Plates = 185.6 lbs. = 54.6 sq. in. r (min.) = 3.73.	14 × 7/16 Plates = 191.5 lbs. = 56.3 sq. in. r (min.) = 3.74.	14 × 1/2 Plates = 197.5 lbs. = 58.1 sq. in. r (min.) = 3.75.	14 × 9/16 Plates = 203.4 lbs. = 59.8 sq. in. r (min.) = 3.76.	14 × 5/8 Plates = 209 4 lbs. = 61.6 sq. in. r (min.) = 3.77.	14 × 11/16 Plates = 215.3 lbs. = 63.3 sq. in. r (min.) = 3.78.	14 × 3/4 Plates = 221.3 lbs. = 65.1 sq. in. r (min.) = 3.78.	14 × 13/16 Plates = 227.2 lbs. = 66.8 sq. in. r (min.) = 3.79.	14 × 7/8 Plates = 233.2 lbs. = 68 6 sq. in. r (min.) = 3 80.
26 and under	327.5	338.0	348.5	359.0	369.5	380.0	390.5	401.0	411.5
28	326.7	337.5	348.5	359.0	369.5	380.0	390.5	401.0	411.5
30	316.7	327.2	337.7	348.3	358.9	369.5	380.0	390.6	401.1
32	206.6	318.0	327.2	337.4	347.7	358.0	368.2	378.5	388.8
34	296.6	306.6	316.6	326.5	336.5	346.5	356.4	366.4	376.4
36	286.7	296.4	306.0	315.7	325.3	335.0	344.7	354.3	364.0
38	276.7	286.0	295.4	304.8	314.2	323.6	332.9	342.3	351.7
40	266.6	275.7	284.8	293.9	303.0	312.1	321.2	330.3	339.3
42	256.6	265.5	274.3	283.0	291.8	300.6	309.4	318.2	327.0
44	246.6	255.2	263.6	272.2	280.6	289.2	297.6	306.1	314.6
46	236.6	244.9	253.0	261.3	269.5	277.7	285.8	294.0	302.3
48	226.7	234.6	242.5	250.4	258.3	266.2	274.1	282.0	290.0
50	216.6	224.3	231.0	239.5	247.1	254.8	262.3	269.9	277.6

14″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6 1/8″ × 3/4″. 1 web plate 8″ × 3/4″. 2 side plates 14″ wide.

Length of Column in Feet.	14 × 3/8 Plates = 197.8 lbs. = 58.2 sq. in. r (min.) = 3.71.	14 × 7/16 Plates = 203.8 lbs. = 59.9 sq. in. r (min.) = 3.72.	14 × 1/2 Plates = 209.7 lbs. = 61.7 sq. in. r (min.) = 3.73.	14 × 9/16 Plates = 215.7 lbs. = 63.4 sq. in. r (min.) = 3.74.	14 × 5/8 Plates = 221.6 lbs. = 65.2 sq. in. r (min.) = 3.75.	14 × 11/16 Plates = 227.6 lbs. = 66.9 sq. in. r (min.) = 3.76.	14 × 3/4 Plates = 233.5 lbs. = 68.7 sq. in. r (min.) = 3.77.	14 × 13/16 Plates = 239.5 lbs. = 70.4 sq. in. r (min.) = 3.77.	14 × 7/8 Plates = 245.4 lbs. = 72.2 sq. in. r (min.) = 3.78.
26 and under	349.1	359.6	370.1	380.6	391.1	401.6	412.1	422.6	433 1
28	347.4	358.3	369.1	380.0	390.9	401.6	412.1	422.6	433.1
30	336.7	347.2	357.9	368.4	378.9	389.5	400.1	410.7	421.2
32	326.0	336.3	346.6	356.8	367.1	377.3	387.6	397.9	408.2
34	315.3	325.2	335.2	345.2	355.1	365.2	375.2	385.1	395.1
36	304.5	314.2	324.0	333.6	343.3	353.0	362.7	372.4	382.0
38	293.8	303.2	312.6	322.0	331.4	340.8	350 2	359.6	369.0
40	283.1	292.2	301.3	310.4	319.5	328.6	337.7	346.8	355.9
42	272.3	281.2	290.0	298.8	307.6	316.4	325.2	334.0	342.8
44	261.6	270.2	278.7	287.2	295.7	304.2	312.7	321.2	329.8
46	250.9	259.1	267.4	275.6	283.8	292.1	300.3	308.5	316.7
48	240.2	248.1	256.1	264.0	272.0	279.8	287.8	295.7	303.6
50	229.5	237.1	244.8	252.4	260.0	267.6	275.3	283.0	290.6

SAFE LOADS IN TONS OF 2000 LBS.

Steel Z-Bar Columns, Square Ends.

Allowed strains per square inch for steel, safety factor 4:
12,000 lbs. for lengths of 90 radii or under.
$17,100-57\frac{l}{r}$ for lengths over 90 radii.

16″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web plate 10″ × 1″. 2 side plates 16″ wide.

Length of Column in Feet.	16 × ½ Plates = 226.7 lbs. = 66.7 sq. in. r (min.) = 4.50.	16 × 9/16 Plates = 233.5 lbs. = 68.7 sq. in. r (min.) = 4.50.	16 × ⅝ Plates = 240.3 lbs. = 70.7 sq. in. r (min.) = 4.50.	16 × 11/16 Plates = 247.1 lbs. = 72.7 sq. in. r (min.) = 4.51.	16 × ¾ Plates = 253.9 lbs. = 74.7 sq. in. r (min.) = 4.51.	16 × 13/16 Plates = 260.7 lbs. = 76.7 sq. in. r (min.) = 4.51.	16 × ⅞ Plates = 267.5 lbs. = 78.7 sq. in. r (min.) = 4.52.	16 × 15/16 Plates = 274.3 lbs. = 80.7 sq. in. r (min.) = 4.52.	16 × 1 Plates = 281.1 lbs. = 82.7 sq. in. r (min.) = 4.52.
32 and under	400.1	412.1	424.1	436.1	448.1	460.1	472.1	484.1	496.1
34	397.7	409.8	421.9	433.9	446.0	458.1	470.2	482.2	494.2
36	387.6	399.3	411.1	422.9	434.7	446.5	458.2	470.0	481.8
38	377.5	388.9	400.4	411.8	423.4	434.8	446.3	457.9	469.3
40	367.3	378.5	389.6	400.9	412.1	423.2	434.4	445.6	456.7
42	357.1	368.0	378.9	389.8	400.7	411.6	422.5	433.4	444.2
44	347.0	357.6	368.2	378.8	389.4	400.0	410.5	421.1	431.7
46	336.9	347.1	357.4	367.7	378.1	388.4	398.6	409.0	419.2
48	326.7	336.7	346.7	356.7	366.7	376.8	386.7	396.7	406.7
50	316.6	326.3	336.0	345.7	355.4	365.1	374.8	384.5	394.2

18″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web plate 12″ × 1″. 2 side plates 18″ wide.

Length of Column in Feet.	18 × ½ Plates = 240.4 lbs. = 70.7 sq. in. r (min.) = 4.71.	18 × 9/16 Plates = 248.0 lbs. = 72.9 sq. in. r (min.) = 4.81.	18 × ⅝ Plates = 255.7 lbs. = 75.2 sq. in. r (min.) = 4.90.	18 × 11/16 Plates = 263.0 lbs. = 77.4 sq. in. r (min.) = 4.98.	18 × ¾ Plates = 271.0 lbs. = 79.7 sq. in. r (min.) = 5.06.	18 × 13/16 Plates = 278.6 lbs. = 81.9 sq. in. r (min.) = 5.14.	18 × ⅞ Plates = 286.3 lbs. = 84.2 sq. in. r (min.) = 5.22.	18 × 15/16 Plates = 293.9 lbs. = 86.4 sq. in. r (min.) = 5.26.	18 × 1 Plates = 301.6 lbs. = 88.7 sq. in. r (min.) = 5.26.
34 and under	424.1	437.6	451.1	464.6	478.1	491.6	505.1	518.6	532.1
36	419.7	436.8	451.1	464.6	478.1	491.6	505.1	518.6	532.1
38	409.4	426.4	443.2	456.2	476.8	491.6	505.1	518.6	532.1
40	399.2	416.0	432.7	449.5	466.0	482.6	499.1	514.2	527.5
42	388.9	405.6	422.3	438.8	455.3	471.7	488.1	503.0	516.0
44	378.7	395.2	411.7	428.2	444.5	460.8	477.0	491.8	504.5
46	368.4	384.9	401.2	417.5	433.8	449.9	466.0	480.5	493.0
48	358.1	374.5	390.7	406.9	423.0	439.0	454.9	469.3	481.4
50	347.9	364.1	380.2	396.2	412.2	428.1	443.9	458.1	469.9

SAFE LOADS IN TONS OF 2000 LBS.

STEEL Z-BAR COLUMNS, SQUARE ENDS.

Allowed strains per square inch for steel, safety factor 4:
12,000 lbs. for lengths of 90 radii or under.
$17,100-57\frac{l}{r}$ for lengths over 90 radii.

20″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web plate 14″ × 1″. Side plates 20″ wide.

Length of Column in Feet.	2 Side Plates.			4 Side Plates.					
	20 × ⅞ Plates = 304.9 lbs. = 89.7 sq. in. r (min.) = 5.24.	20 × 15/16 Plates = 313.4 lbs. = 92.2 sq. in. r (min.) = 5.32.	20 × 1 Plates = 321.9 lbs. = 94.7 sq. in. r (min.) = 5.39.	20 × 1 1/16 Plates = 330.4 lbs. = 97.2 sq. in. r (min.) = 5.44.	20 × 1⅛ Plates = 338.9 lbs. = 99.7 sq. in. r (min.) = 5.50.	20 × 1 3/16 Plates = 347.4 lbs. = 102.2 sq. in. r (min.) = 5.55.	20 × 1¼ Plates = 355.9 lbs. = 104.7 sq. in. r (min.) = 5.60.	20 × 1 5/16 Plates = 364.4 lbs. = 107.2 sq. in. r (min.) = 5.65.	20 × 1⅜ Plates = 372.9 lbs. = 109.7 sq. in. r (min.) = 5.69.
38 and under	538.1	553.1	568.1	583.1	598.1	613.1	628.1	643.1	658.1
40	532.9	551.1	568.1	583.1	598.1	613.1	628.1	643.1	658.1
42	521.2	539.2	557.2	574.5	591.9	609.0	626.4	643.1	658.1
44	509.5	527.3	545.3	562.3	579.4	596.5	613.7	630.7	648.0
46	497.7	515.5	533.3	550.1	567.0	583.8	600.9	617.8	634.8
48	486.1	503.6	521.2	538.0	554.6	571.2	588.1	604.8	621.6
50	474.4	491.8	509.2	525 7	542.2	558.6	575.2	591.8	608.4

20″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web plate 14″ × 1″. 4 side plates 20″ wide.

Length of Column in Feet.	20 × 1 7/16 Plates = 381.5 lbs. = 112.2 sq. in. r (min.) = 5.74.	20 × 1½ Plates = 390.0 lbs. = 114.7 sq. in. r (min.) = 5.79.	20 × 1 9/16 Plates = 398.5 lbs. = 117.2 sq. in. r (min.) = 5.83.	20 × 1⅝ Plates = 407.0 lbs. = 119.7 sq. in. r (min.) = 5.88.	20 × 1 11/16 Plates = 415.5 lbs. = 122.2 sq. in. r (min.) = 5.92.	20 × 1¾ Plates = 424.0 lbs. = 124.7 sq. in. r (min.) = 5.93.	20 × 1 13/16 Plates = 432.5 lbs. = 127.2 sq. in. r (min.) = 5.93.	20 × 1⅞ Plates = 441.0 lbs. = 129.7 sq. in. r (min.) = 5.93.	20 × 1 15/16 Plates = 449.5 lbs. = 132.2 sq. in. r (min.) = 5.93.
42 and under	673.1	688.1	703.1	718.1	733.1	748.1	763.1	778.1	793.1
44	665.0	682.5	699.7	717.0	733.1	748.1	763.1	778.1	793.1
46	651.7	668.8	686.0	703.1	720.2	735.6	750.2	764.7	779.3
48	638.4	655.3	672.2	689.2	706.1	721.2	735.5	749.8	764.1
50	625.0	641.7	658.4	675.3	692.0	706.8	720.8	734.8	748.8

SAFE LOADS IN TONS OF 2000 LBS.

STEEL Z-BAR COLUMNS, SQUARE ENDS.

Allowed strains per square inch for steel, safety factor 4:

12,000 lbs. for lengths of 90 radii or under.

$17,100-57\frac{l}{r}$ for lengths over 90 radii.

20″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web plate 14″ × 1″. 6 side plates 20″ wide.

Length of Column in Feet.	20 × 2 Plates = 458.0 lbs. = 134.7 sq. in. r (min.) = 5.92.	20 × 2$\frac{1}{16}$ Plates = 466.5 lbs. = 137.2 sq. in. r (min.) = 5.92.	20 × 2⅛ Plates = 475.0 lbs. = 139.7 sq. in. r (min.) = 5.92.	20 × 2$\frac{3}{16}$ Plates = 483.5 lbs. = 142.2 sq. in. r (min.) = 5.92.	20 × 2¼ Plates = 492.0 lbs. = 144.7 sq. in. r (min.) = 5.91.	20 × 2$\frac{5}{16}$ Plates = 500.5 lbs. = 147.2 sq. in. r (min.) = 5.91.	20 × 2⅜ Plates = 509.0 lbs. = 149.7 sq. in. r (min.) = 5.91.	20 × 2$\frac{7}{16}$ Plates = 517.5 lbs. = 152.2 sq. in. r (min.) = 5.91.	20 × 2½ Plates = 526.0 lbs. = 154.7 sq. in. r (min.) = 5.90.
44 and under	808.1	823.1	838.1	853.1	868.1	883.1	898.1	913.1	928.1
46	793.7	808.3	823.0	837.5	852.1	866.7	881.2	895.8	910.4
48	778.2	792.5	806.9	821.2	835.5	849.7	864.0	878.3	892.6
50	762.6	776.7	790.8	804.7	818.7	832.8	846.7	860.7	874.7

20″ STEEL Z-BAR COLUMNS.

Section: 4 Z-bars 6⅛″ × ⅞″. 1 web plate 14″ × 1″. 6 side plates 20″ wide.

Length of Column in Feet.	20 × 2$\frac{9}{16}$ Plates = 534.5 lbs. = 157.2 sq. in. r (min.) = 5.90.	20 × 2⅝ Plates = 543.0 lbs. = 159.7 sq. in. r (min.) = 5.90.	20 × 2$\frac{11}{16}$ Plates = 551.5 lbs. = 162.2 sq. in. r (min.) = 5.90.	20 × 2¾ Plates = 560.0 lbs. = 164.7 sq. in. r (min.) = 5.90.	20 × 2$\frac{13}{16}$ Plates = 568.5 lbs. = 167.2 sq. in. r (min.) = 5.89.	20 × 2⅞ Plates = 577.0 lbs. = 169.7 sq. in. r (min.) = 5.89.	20 × 2$\frac{15}{16}$ Plates = 585.5 lbs. = 172.2 sq. in. r (min.) = 5.89.	20 × 3 Plates = 594.0 lbs = 174.7 sq. in. r (min.) = 5.89.
42 and under	943.1	958.1	973.1	988.1	1003.1	1018.1	1033.1	1048.1
44	943.1	958.1	973.0	987.8	1002.5	1017.5	1032.3	1047.3
46	925.0	939.6	954.2	968.8	983.3	997.7	1012.3	1026.8
48	966.9	921.1	935.4	949.6	963.9	978.1	992.3	1006.5
50	888.7	902.6	916.6	930.5	944.5	958.4	972.4	986.1

Phœnix Columns.—To determine the value of Phœnix columns under loads, from tests, the following formula has been adopted:

$$\frac{P}{S} = \frac{42,000}{1 + \frac{l^2}{50,000r^2}}.$$

The expression $\frac{P}{S}$ represents the $\frac{\text{total load in pounds}}{\text{sectional area in square inches}}$; or, in other words, the crushing strain per square inch of section; l is the length in feet between bearings, and r is the least radius of gyration.

Applying the above formula to the several patterns of segmental columns, the table of allowable working strains per square inch of section has been prepared; the allowable working strains being in each case about *one fourth* of the ultimate strength of the column.

SAFE LOADS FOR PHŒNIX COLUMNS, IN POUNDS PER SQUARE INCH OF SECTIONAL AREA.

SQUARE-END BEARINGS.

Length in Feet.	Col. *A*.	Col. *B*1.	Col. *B*2.	Col. *C*.	Col. *E*.	Col. *G*.
10	9,323	9,833	10,024	10,195	10,351	10,411
12	8,885	9,564	9,830	10,067	10,288	10,371
14	8,420	9,267	9,607	9,924	10,215	10,326
16	7,943	8,944	9,364	9,783	10,131	10,275
18	7,463	8,610	9,105	9,575	10,037	10,216
20	6,997	8,260	8,830	9,386	9,935	10,152
22	6,526	7,906	8,541	9,185	9,824	10,082
24	6,090	7,550	8,250	8,973	9,705	10,005
26	......	7,201	7,955	8,755	9,580	9,926
28	......	6,860	7,660	8,527	9,450	9,841
30	......	6,527	7,366	8,297	9,314	9,750
32	......	......	7,075	8,070	9,170	9,654
34	......	......	......	7,837	9,021	9,555
36	......	......	......	7,604	8,870	9,441
38	......	......	......	7,375	8,717	9,341
40	......	......	......	7,147	8,561	9,235

TABLE OF DIMENSIONS OF PHŒNIX COLUMNS.

The dimensions given in the following table are subject to slight variations, which are unavoidable in rolling iron shapes.

The weights of columns given are those of the 4, 6, or 8 segments of which they are composed. The *shanks* of the rivets used in joining the segments together only make up the quantity of metal removed in making the holes, but the *rivet-heads* add from 2 to 5 per cent to the weights given. The rivets are spaced 3, 4, or 6 inches apart from centre to centre, and somewhat more closely at the ends than toward the centre of the column.

Any desired thickness between the minimum and maximum for any given size can be furnished. *G* columns have 8 segments, *E* columns 6 segments, *C*, B^2, B^1, and *A* have 4 segments.

Least radius of gyration equals $D \times .3636$.

One Segment.		Diameters in Inches.			One Column.			Size of Rivets.
Thickness in Inches.	Weight in Lbs. per Yard.	*d* Inside.	*D* Outside.	D^1 Over Flanges.	Area of Cross-section. Sq. In.	Weight per Foot in Pounds.	Least Radius of Gyration in Ins.	
3/16	9 1/2	A—3 5/8	4	6 1/16	3.8	12.6	1.45	3/8 × 1 1/8
1/4	12		4 1/8	6 3/16	4.8	16.0	1.50	1 1/4
5/16	14 1/2		4 1/4	6 5/16	5.8	19.3	1.55	1 3/8
3/8	17		4 3/8	6 7/16	6.8	22.6	1.59	1 1/2
1/4	16	B^1—4 13/16	5 5/16	8 1/16	6.4	21.3	1.92	1/2 × 1 5/8
5/16	19 1/2		5 7/16	8 1/8	7.8	26.0	1.96	1 3/4
3/8	23		5 9/16	8 1/4	9.2	30.6	2.02	1 3/4
7/16	26 1/2		5 11/16	8 3/8	10.6	35.3	2.07	1 7/8
1/2	30		5 13/16	8 7/16	12.0	40.0	2.11	1 7/8
9/16	33 1/2		5 15/16	8 1/2	13.4	44.6	2.16	2
5/8	37		6 1/16	8 5/8	14.8	49.3	2.20	2 1/8
1/4	18 1/2	B^2—5 15/16	6 7/16	9 1/8	7.4	24.6	2.34	1/2 × 1 5/8
5/16	22 1/2		6 9/16	9 1/4	9.0	30.0	2.39	1 3/4
3/8	26 1/2		6 11/16	9 5/16	10.6	35.3	2.43	1 3/4
7/16	30 1/2		6 13/16	9 3/8	12.2	40.6	2.48	1 7/8
1/2	34 1/2		6 15/16	9 1/2	13.8	46.0	2.52	1 7/8
9/16	38 1/2		7 1/16	9 5/8	15.4	51.3	2.57	2
5/8	42 1/2		7 3/16	9 11/16	17.0	56.6	2.61	2 1/8

Least radius of gyration equals $D \times .3636$—(*continued*).

One Segment.		Diameters in Inches.			One Column.			
Thickness in Inches.	Weight in Lbs. per Yard.	d Inside.	D Outside.	D^1 Over Flanges.	Area of Cross-section. Sq. In.	Weight per Foot in Pounds.	Least Radius of Gyration in Ins.	Size of Rivets.
1/4	25	C—7 3/16	7 11/16	11 9/16	10.0	33.3	2.80	5/8 × 1 7/8
5/16	30		7 13/16	11 5/8	12.0	40.0	2.85	2
3/8	35		7 15/16	11 11/16	14.0	46.6	2.90	2 1/8
7/16	40		8 1/16	11 3/4	16.0	53.3	2.94	2 1/4
1/2	45		8 3/16	11 13/16	18.0	60.0	2.98	2 3/8
9/16	48		8 5/16	11 7/8	19.2	64.0	3.03	2 1/8
5/8	53		8 7/16	12	21.2	70.6	3.08	2 1/8
11/16	58		8 9/16	12 1/16	23.2	77.3	3.12	3 × 2 5/8
3/4	63		8 11/16	12 3/16	25.2	84.0	3.16	2 3/4
13/16	68		8 13/16	12 5/16	27.2	90.6	3.21	2 7/8
7/8	73		8 15/16	12 7/16	29.2	97.3	3.26	3
1	83		9 3/16	12 9/16	33.2	110.6	3.34	2 3/4
1 1/8	93		9 7/16	13 1/4	37.2	124.0	3.43	2 7/8
1 1/4	103		9 11/16	12 13/16	41.2	137.3	3.52	3
1/4	28	E—11	11 1/2	15 7/16	16.8	56	4.18	5/8 × 2
5/16	32		11 5/8	15 9/16	19.2	64	4.23	2 1/8
3/8	36		11 3/4	15 11/16	21.6	72	4.28	2 1/8
7/16	40		11 7/8	15 13/16	24.0	80	4.32	2 1/4
1/2	44		12	15 7/8	26.4	88	4.36	2 3/8
9/16	48		12 1/8	16	28.8	96	4.40	2 3/8
5/8	53		12 1/4	16 1/16	31.8	106	4.45	2 1/8
11/16	58		12 3/8	16 3/16	34.8	116	4.50	3/4 × 2 3/8
3/4	63		12 1/2	16 5/16	37.8	126	4.55	2 3/8
13/16	68		12 5/8	16 7/16	40.8	136	4.60	2 1/2
7/8	73		12 3/4	16 5/8	43.8	146	4.64	2 3/4
1	83		13	16 3/4	49.8	166	4.73	2 7/8
1 1/8	93		13 1/4	17	55.8	186	4.82	3
1 1/4	103		13 1/2	17 3/16	61.8	206	4.91	3 1/8
5/16	30	C—14 3/8	15	19 1/8	24	80.0	5.45	5/8 × 2
3/8	35		15 1/8	19 1/4	28	93.3	5.50	2
7/16	40		15 1/4	19 3/8	32	106.6	5.55	2 1/8
1/2	45		15 3/8	19 7/16	36	120.0	5.59	2 1/4
9/16	50		15 1/2	19 1/2	40	133.3	5.63	2 3/8
5/8	55		15 5/8	19 5/8	44	146.6	5.68	2 1/2
11/16	60		15 3/4	19 3/4	48	160.0	5.72	3/4 × 2 3/8
3/4	65		15 7/8	19 7/8	52	173.3	5.77	2 5/8
13/16	70		16	20	56	186.6	5.82	2 3/4
7/8	75		16 1/8	20 1/8	60	200.0	5.87	2 7/8
1	85		16 3/8	20 3/8	68	226.6	5.95	3
1 1/8	95		16 5/8	20 5/8	76	253.3	6.04	3 1/8
1 1/4	105		16 7/8	20 3/4	84	280.0	6.14	3 1/4
1 3/8	115		17 1/8	21	92	306.6	6.23	3 3/8

CHAPTER III.

COLUMN CONNECTIONS.

Column Connections.—It was previously mentioned that the advantages of different shape columns for connections

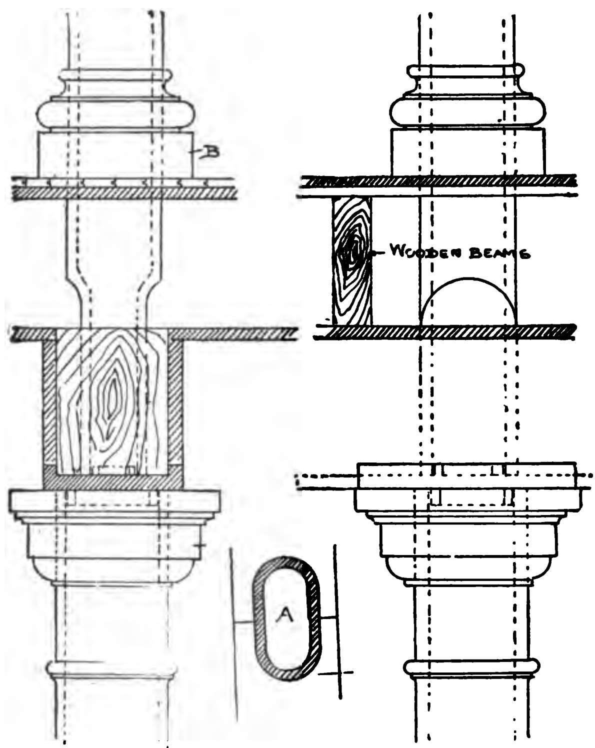


FIG. 31.

with the floor girders, wall girders, and with each other perform an important part in their selection; in fact, it is so

important that the entire strength and rigidity of the structure depends upon these connections.

Cast-iron columns with wooden girders, as in Fig. 31, were used extensively at first for the interior columns of buildings; in fact, are used to a considerable extent at the present time.

By glancing at the details it will be seen that bolts or rivets are not used in any manner whatever, and the equilibrium depends solely upon the wooden girders and imposed weight holding the entire connection in place, and, as one writer has stated, "much the same as a child would pile blocks up and steady the pile with its hands." Then again the columns are also connected to each other in the same style of construction by flanges as shown in Fig. 32.

In the first detail the pintle *A* is cast as a part of the upper columns, and the shape as shown. In the second, the lower column remains the same diameter, and the wooden girders are cut to fit the shape of the column. The columns are

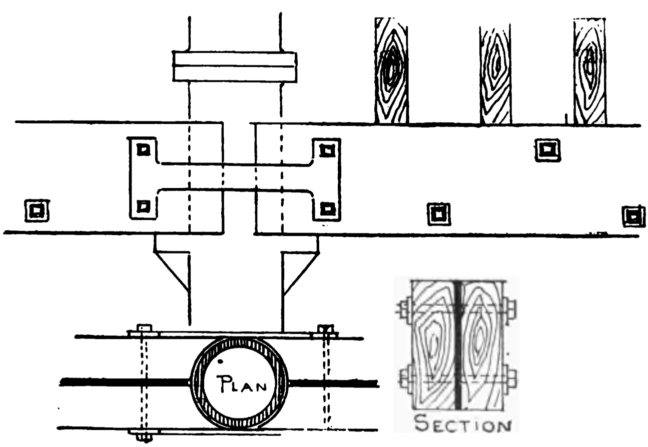


FIG 32.

secured to each other by bolts through their flanges, and the wooden girders are secured by straps placed each side, as shown in the detail.

The change from wooden floor beams and girders to iron

and steel brought about some alterations in the general details of the columns, which is shown in Fig. 33. Iron beams were

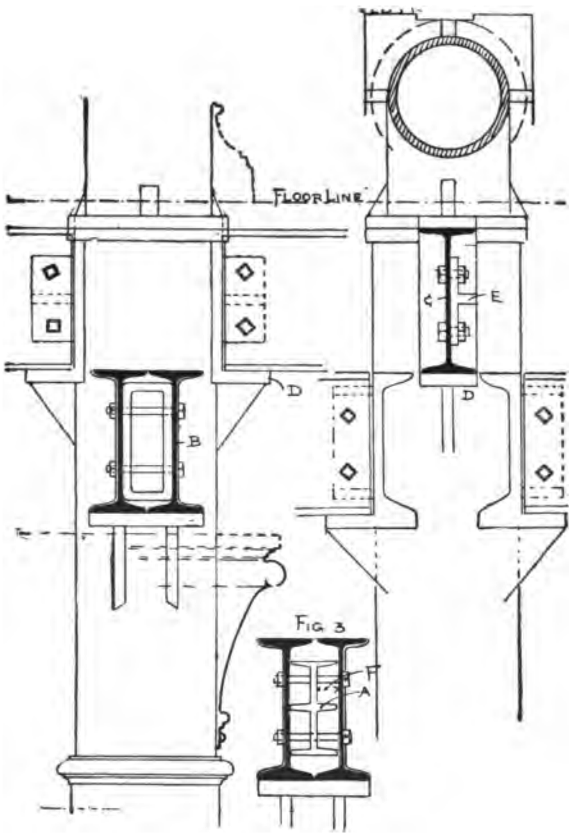


FIG. 33.

placed side by side, as shown at *B*, in place of the wooden girders; then secured to the column by bolts to cast-iron lugs, forming at the same time a separator in the girder.

These separators were made in two ways—one cast solid, as at *B*, the other as shown at *A*, Fig. 3, and the girders rest upon brackets cast with the column. The floor beams are secured to the columns by bolts in the same manner as the girders,

and also rest upon brackets. The lugs and brackets are cast about the same thickness as the column, and project 5 or 6 inches from the body. This later detail, Fig. 33, is generally adopted as the connections for all interior cast-iron columns, and the joints, if covered by a cast-iron base, can be at any distance above the floor beams.

Cast-iron Column Connections in the Skeleton Frame. —In changing from the ordinary method of building to the skeleton frame, the joints of the columns were altered somewhat, and the beams and girders when cast-iron columns are used should be secured by wrought-iron knees and bolts, as shown at Fig. 34. The columns were also changed from

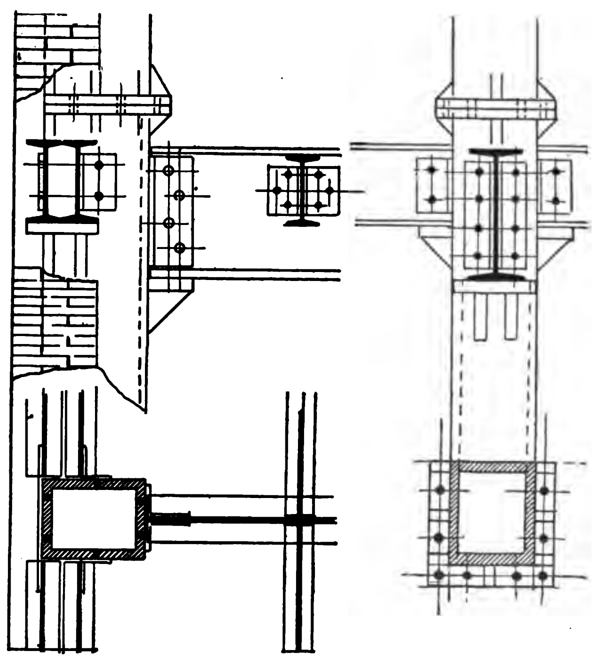


FIG. 34

circular to square to allow the masonry to fit square with the column. This form provides a simpler connection for the

curtain wall and floor girders than the circular shape. The flanges of the joint are reinforced by small brackets, as shown.

The curtain-wall girders are secured to the side of the column by knees and bolts, and further secured by straps placed at the back and extending to the girders on the opposite side.

This system of bolting the floor girders to the cast-iron columns is to be preferred to that of depending upon lugs cast with the column, as in Fig. 33, for the reason that any number of bolts can be placed in the head to make a rigid connection when the joint especially is above the girders.

The same system could be used for the curtain-wall girders; and instead of using those made up of beam sections, plate or lattice girders could be adopted.

The weakest point of the connection is at the joint of the column, and depends solely upon the strength of the bolts in the flanges.

Z-bar Column Connections.—A material change has been effected from cast-iron columns to those made up of rolled shapes. In these rolled material columns numerous sections are formed by riveting together angles, plates, channels, I-beams, Z-shapes, etc., or of some patented shape which form segments of a circular section such as herein described; and, as previously mentioned, the requirement is that the column used shall be well adapted for connection.

It is well known that metal near the neutral axis of a column is not of much value, and that a proper disposition of metal farthest from the neutral axis is best effected in the cylindrical section. Therefore, the unit strength of this section is somewhat greater in long columns than that of others. For proportion of length to diameter, which occurs in the majority of buildings, there is little or no difference in strength among the various sections as mentioned above.

To overcome the bending tendency of the column caused

by eccentric loading, the floor and wall girders should be applied as close as possible to the neutral axis.

The advocates of the Z-bar column, a section made up of four Z-bars and a plate, as shown in Fig. 35, claim that it is

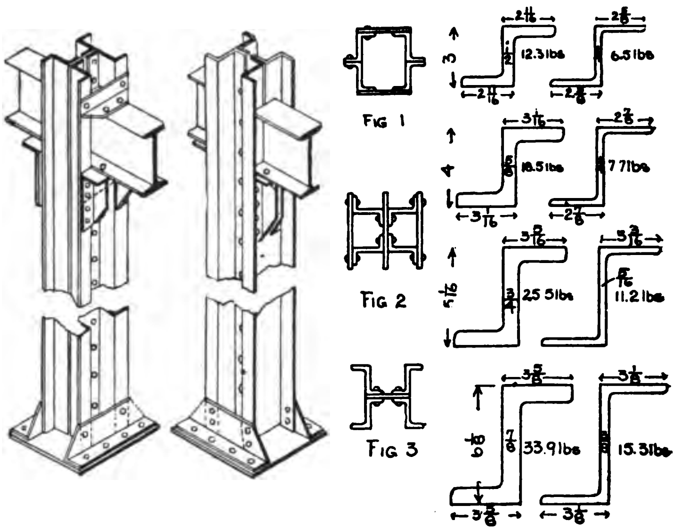


FIG. 35.

probably the best section which meets the general requirements for the construction of buildings.

This section, they claim, combines a minimum of shop-work with good adaptation for connection and accessibility of all its surfaces. The sketch shows a single beam entering between the Z-bars almost to the very centre of the column, and secured by rivets through the top and bottom flanges; it also rests upon a heavy bracket made up of angles and riveted to the outer legs of the Z-bars.

A beam can also be placed between the legs of the Z-bars at right angles to the one that is shown, and supported upon brackets riveted to the middle web of the Z-bars.

The connection of one column to another is not shown in

this figure, but is shown better in Fig. 36. The columns are separated by a plate made of various thicknesses, depending in a great measure upon the load transmitted to the column from the floor girders. The curtain-wall girders are placed higher than the bottom of the latter and rest upon cast-iron blocks as at *A*, which are secured to the plate by bolts passing

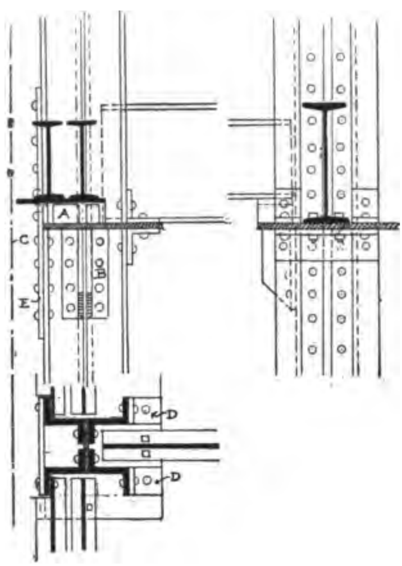


FIG. 36.

through the flange of the beams, then through the cast blocks and through the plate. Stiffness and strength is given to the plate under this girder by angle brackets riveted to the body of the column, as at *B*. The plate is also reinforced under the floor girder by angle-knees, which also serve to join the columns together, with rivets passing through this lower and upper knee, as at *D*. The columns are again stiffened at the back by a splice plate, riveted by any number of rivets, and shown at *E*.

The dotted line *C* represents the party wall of the building. If this connection is to be used on the inner columns of a building, the plates between the columns will extend to receive a beam or girder on the opposite side; then the splice plate is not required.

When the Z-bar column stands clear of any wall, and is placed to carry beams and girders at right angles to each other, being open on four sides, so that every beam or girder may enter between the flanges, a stiffer connection could not be desired. In fact, the entire load is concentrated near the centre, and every gain in this direction adds greatly to the efficiency of the column; and one in which a one-sided load can be applied close to the axis is good for a much greater unit strain than where the application must be made farther from the centre.

The joint of these Z-bar columns is frequently made at the top of the girders, a plate also being used to make the separation; then heavy brackets are riveted to the body of the column, as shown at *B*, to support all beams and girders.

Phœnix Column Connections.—The makers of the Phœnix column claim, among the many advantages of that section, that the means for applying the loads closely to the axis of the column is an advantage which places it among the many desirable sections to use.

For instance, if the load is applied to the shell of the column at one side, this load travels around the column and downward, at an angle of about 45°, so that, at a distance below the load equal to the diameter of the column, the whole column will receive an equal load on all parts.

There are several methods of making connection to the flanges. In some cases, where filler bars are used between the flanges, the load can be transmitted directly to the opposite flange by means of a gusset plate, or the filler bars can be forged out to form a connection. In a four-segment column

four loads can be applied at the four connecting flanges, in a six-segment column six, an eight-segment column eight.

In order to make a desirable connection to any column, the bracket holes must fit the holes in the column exactly, and the rivets must completely fill the holes. To carry any great load, the brackets must necessarily extend a great distance below the seat in order to *get enough rivets in to take the shear.* This is not the case with the filler-bar, it extends the full height of column.

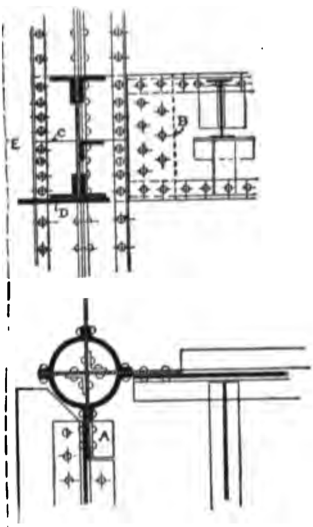


FIG. 37.

In Fig. 37 it will be noticed that the cross-pintle extends to *B*, or is simply a continuation of the web of the floor girder carried clear through the column, distributing the load to all parts of the column, and overcoming in a great measure the tendency of eccentric loading.

The wall girders are carried in the same manner and riveted to the pintles shown in plan view. Those which carry the wall girders are riveted to the floor-girder pintle by angles the full height of the pintles.

The joint of the columns is at *C*, plates between being dispensed with. By means of these cross-pintles it is possible to make a continuous column from cellar to roof in which the joints are actually stronger than the body of the column.

By referring to the detail (Fig. 37) it will be noticed that the wall girder is composed of angles and plates similar in construction to the floor girder; an angle is riveted to the side level with the bottom of the floor beam. This is to receive the floor arch. A plate *D* is also riveted to the bottom of the

girder, so that the curtain wall may be supported at each story. The entire distance from the party line E to the inside angle of girder being equal to 12 inches, is supported upon a plate 11 inches wide, if the wall is 16 inches the plate is increased in width and the girders in sectional area. These plates are generally ⅜ of an inch thick and riveted to the angles with rivets about 6 inches centres. This section of wall girder may be applied to any column or number of columns; in fact, the plate girder and lattice girder seem to be the best section that could be used. The lattice section is so arranged that the wall is built enclosing it completely. Angles are also riveted to the latticing to receive the floor arches.

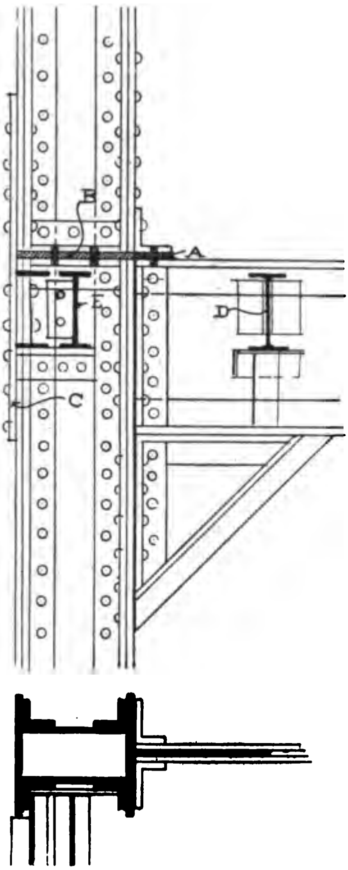


FIG. 38.

Connections of Column Sections Made up of Angles and Plates.—Girders of built sections of plate and angles make more efficient connection than those made up of rolled beams. Columns built of the same sections are more readily joined together at the floor levels, and with each other than any other form, they give a great amount of rigidity, and are recommended to be used, especially when the arrangement of the doors and other openings which occur in the partitions and outer walls are such that it would be impossible to devise a system of lateral bracing. The stiffness of the individual

column in a skeleton-framed structure—or in any building construction—is an element of resistance of considerable value if the connections are rigid. If it is impossible to apply lateral bracing to the frame work, the columns should be joined together by complete splice plates on all sides that have no beam or girders connected; these splice-plates could be used to advantage between the columns and the girders.

An ideal column, therefore, is one in which each column is a unit throughout the entire height of the building, which condition is possible when they are made up as shown in Figs. 36 to 40.

A considerable degree of security against injury from any cause to which buildings are subjected can be obtained when the columns are constructed in this manner; in fact, the joints at the floor level are stronger than the body of the column.

Columns constructed as shown in Figs. 36 to 40 can be made to any practical length, some of which are further illustrated, where the same section is carried through three stories making the columns nearly 40 feet in length.

Where so much depends upon the columns as in the skeleton frame, every precaution against accidents should be taken.

The rolled iron and steel, before the members are riveted together, should have these members inspected singly, for the quality of the material and for surface, and then the finished column inspected for workmanship; this offers a guaranty against any serious failure.

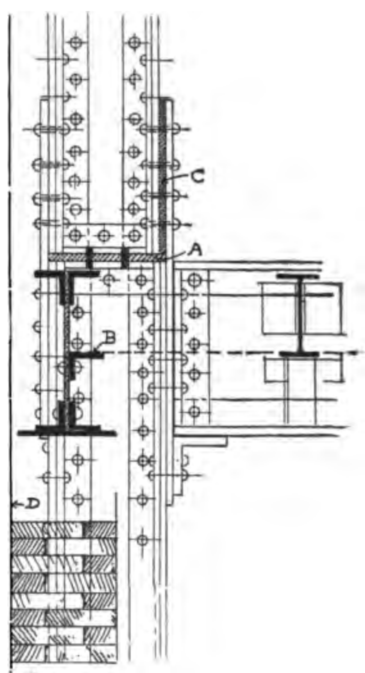


FIG. 39.

The connection shown at Fig. 38, of the box column made up of two webs, two cover plates, and joined together by angles, may be applied to single web, Z-bar, or any rectangular sections.

In joining the upper and lower columns, they are first separated by the plate *B*, angle-knees being riveted top and bottom, through which holes for bolts or rivets are punched. An angle-knee is also placed on the inside face over the floor girder. At the back a splice plate *C*, the full width of the covers, is placed, extending at least 2 feet from the joint. When the work is riveted a stiff and rigid joint is secured.

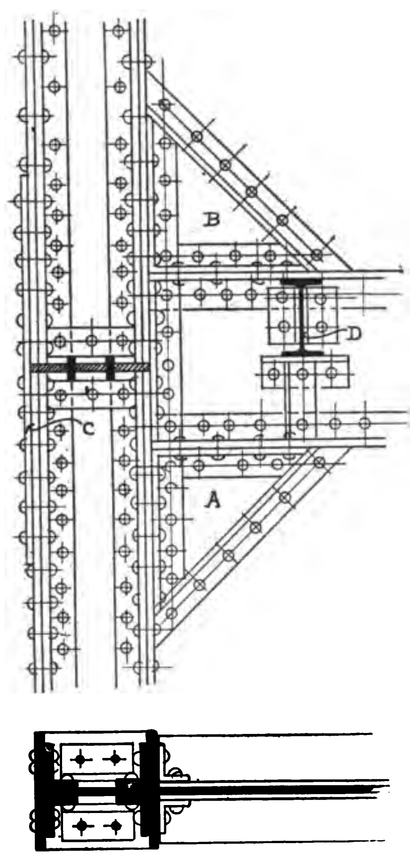


FIG. 40.

This same connection may be improved upon in the manner shown by Fig. 39. When the column sections are decreased at certain floor levels a filler plate will be required, as shown at *C*. The joint of the column is more rigid in this connection, but to prevent any lateral displacement of the floor girders and columns the knee-braces are preferred. When high and narrow buildings are designed the ideal joint is that shown at Fig. 40, the plate separating the upper and lower column being placed at the centre

of floor girder; the knee-brace *A* placed at the ceiling and the brace *B* at the floor level.

The curtain-wall girder, if made of I-beams, will be level with the bottom of floor beam *D*; if made of a plate or latticing it can be secured to the lower and upper column much the same as shown in Fig. 39, a portion of the girder being cut out to pass the connection plate.

The position of the columns in the building using such joints will be determined in a great measure by the partitions, then the knee-braces will not be seen. If such is impossible, the floor brace *B* may be dispensed with, and the inside splice plate, as shown in Fig. 39, be adopted.

All connections as described are to be riveted at the works and building with hot wrought iron or wrought steel rivets, thus insuring more rigidity against wind pressure than can be obtained with bolt connections.

Rivet Spacing in Column Joints.—The rivet spacing in all the above details is determined by certain fixed laws. The rivets connecting the girders with the columns depend upon the load to be supported. For example, if there is 27 tons (54,000 lbs.) to be supported at one end of a floor girder and $\frac{7}{8}''$ diameter rivets are used,—the shearing strain being measured on the area of the cross-section and allowing 7500 pounds for wrought iron and steel rivets,—the area of a rivet $\frac{7}{8}$ of an inch in diameter is 0.6013 square inches. This multiplied by 7500 pounds, the safe shearing, $=$ 4510 pounds, the safe amount of strain each rivet can sustain without shearing; dividing 5400 by this we get 12 rivets to support the girder.

If knee-braces are used, a portion of these 12 rivets can be counted in as those supporting the girder.

The rivets in the column are generally spaced closer at the joints, say 3 inches for $\frac{3}{4}''$, and 4 inches for $\frac{7}{8}''$ diameter rivets; for the body of the column they should be spaced at a maximum of 6 inches.

If there is more than one cover plate over $\frac{5}{8}''$ thick each,

$\frac{7}{8}''$ in diameter rivets should be used; less than that, use $\frac{3}{4}''$ rivets. If the thickness of plates and angles equals 3 inches, use 1'' diameter rivets.

The rivets in the splice plates are determined by those in the column; in the knee-braces by the girder and column rivets.

The rivets connecting the angle-knees of the top and bottom column through the joint plate are of the same diameter as the rest of the lower column.

CHAPTER IV.

FLOOR LOADS AND FLOOR FRAMING.

EQUALLY important in the construction of the skeleton frame as the columns and column connections is the arrangement of the floor beams and floor girders.

Very many mistakes are undoubtedly due to errors in the calculation of the floor loads.

The arrangements must be such that the material is used in the most economical manner; every member must be calculated. There must be sufficient material, no more nor less; for it is essential not only from economy, but also to reduce the weights of the dead loads on the foundations, and the construction should be as light as consistent with perfect stability.

Dead Loads.—All materials used as a part of the construction of the building are rated as *dead loads;* that is, the floor-beams, girders, arches, columns, walls, flooring, water in tanks, machinery, partitions, plastering, and anything actually a part of the building.

Live Loads.—The weight of persons, office furniture, or stores of any kind that can be moved or changed are usually classed as *live loads.*

For such weights as is usual to apply to the floors, the New York Building Law of 1892 is a good guide to follow: "SEC. 483. In every building used as a dwelling-house, tenement-house, apartment-house, or hotel, each floor shall be of sufficient strength in all its parts to bear safely, upon every superficial foot of its surface, 70 pounds; and if to be used for office purposes

not less than 100 pounds upon every superficial foot; if to be used as a place of public assembly, 120 pounds; and if used as a store, factory, warehouse, or for any other manufacturing or commercial purpose, 150 pounds and upward upon every superficial foot; and every floor shall be of sufficient strength to bear the weight to be imposed thereon in *addition to the weight of the materials of which the floor is composed.* The roofs of all buildings shall be proportioned to bear safely 50 pounds upon every superficial foot of their surface, in addition to the weight of materials composing the same."

In several buildings used as offices the author has calculated the dead loads, and found the average weight to be 100 pounds per square foot. This included the terra-cotta arches eight inches in depth, sleepers, wooden floors, beams, girders, partitions, and plastering on partitions and ceiling; the last two items being actually calculated, and then rated so much per square foot of floor surface.

The usual practice in New York is to make the upper floors of office buildings carry the minimum weight, 70 or 75 pounds, as required by the New York Building Law, and then increase the weight upon the lower floors, say from 75 pounds; for all stories above the third, to 150 pounds upon the first story and basement.

One 12-story skeleton-constructed building in particular in New York the live load upon every floor excepting the roof has a calculated area of 350 pounds per square foot. The building is used as a printing establishment; it is therefore likely that every floor, or portion, will at some time or other be loaded to that extent. The greatest weight, according to the dead and live load supported by the lower columns, is 800 tons.

Chicago's Practice Relating to the Calculations of the Dead and Live Load upon the Floors.—It seems to be the practice in Chicago's high buildings, in regard to the floor loads, to calculate all the beams for the total dead and live

loads, while the girders are required to carry the dead load and about 80 per cent. of the live load, and the columns the dead load and half, or even less, of the live load.

This practice is based on the theory that it is quite possible the beams will some time have to carry all the live load, while the chances are increasingly less that the girders and columns will ever be required to do so.

Take, for example, the Venetian Building, Chicago. The dead weight on the office floors is 100 pounds per square foot; the live loads on the floors above the fourth is taken at 35 pounds per square foot. On the second, third, and fourth floors it is taken at 60 pounds, and on the first floor at 80 pounds. The whole of the dead load and about one half the live load is carried to the columns. The building is 12 stories, the greatest load on the lower columns being about 327 tons.

The Fair Building, Chicago, is 16 stories in height, and the beams above the fifth story are calculated to carry 75 pounds per square foot of live load, the fifth story 130 pounds, the fourth 200 pounds, and all below the fourth, including the first story, 130 pounds.

The floor beams are calculated to carry all the dead load plus the full amount of the live load designated as the maximum for said story.

The girders are calculated to carry all the dead load plus 90 per cent. of the live load designated for said story.

The columns are calculated to carry all the dead loads plus 45 per cent. of the live load on first story, and increases on each story, from that to the sixteenth story, where it is 90 per cent. —an average of about 64 per cent. throughout the building.

It would be good practice and within the limit of safety to have for each story:

The beams	sustain	dead load	+ live load.		
girders	"	"	" + 85 per cent. of live load.		
columns	"	"	" + 75	"	" " "

Floor Framing.—In designing the floor framing of a building the beams and floor girders should be arranged to be strained up to the allowable fibre strain, and if the positions of the columns were fixed according to this arrangement, much economy of material would be gained.

The proper way is to fix upon the loads which the floor-beams must carry per square foot of floor area; that is, the dead and live loads. Then the spans determined by the loads which will strain the beams to the allowed fibre strain.

Suppose, for example, the columns in the side walls of the skeleton frame are spaced 20 feet from centre to centre, and the beams 5-feet centres, this being a practicable distance for the floor arches, and an equal spacing between side walls.

The dead and live load to be carried by the beams is 225 pounds per square foot of floor area.

The load upon the beam would be $5 \times 20 \times 225 = 22{,}500$ pounds, whose coefficient is $20 \times 22{,}500 = 450{,}000$ pounds.

By referring to the table of properties of steel beams, page 69, the coefficient corresponding to this would be a $12'' \times 40$ pound per foot I, whose coefficient is 500,100. This is an excess of strength, and if $12'' \times 32$ pounds per foot I were used, whose coefficient is 395,000 pounds, there would be too little strength.

To accommodate this gain and loss, one of two things must be done, viz., the column centres or the depth of beams be changed.

If we increase the column centre to 21 feet, the total load would be $5 \times 21 \times 225 = 23{,}625$, and the coefficient corresponding to this would be $21 \times 23{,}625 = 496{,}125$ pounds.

If a deeper beam is used, say a 15×41 pounds per foot I, the coefficient by the table is 603,200 pounds—a still greater excess of strength, and the former should be adopted.

It is therefore more economical to space the columns to accommodate the full strength of the beams.

It will be found in working out these floor beams that the

deeper beam is more economical not only for strength, but for stiffness. If thin floors are not required deep beams should be used; then the arches become heavier, the filling above the arches becomes considerable, and if this is of concrete the dead load will have to be increased.

It is therefore desirable that a few trials be given to this important question before its final settlement.

Rolled solid sections should be used in preference to the built-up girders, for these section beams are rolled as deep as 24 inches, with $7\frac{1}{4}$-inch flanges.

In the skeleton frame, or in narrow buildings, the girders generally extend parallel with the narrow front and the beams at right angles.

Having determined the load per square foot to be supported, the following tables will aid the designer in the contruction of the floors:

To Determine Coefficient for Beams.—The following formula for uniform weights gives coefficient for 12,000 pounds strain:

$$\tfrac{1}{8}WL = 12{,}000\frac{I}{e},$$

where W = weight in pounds uniformly distributed
L = length in inches;
I = moment of inertia;
e = distance of extreme lamina from neutral axis (half the depth of I-beam);
C = coefficient.

$$WL = 96{,}000\frac{I}{e};$$

or if L be given in feet as is usual, then

$$WL = 8000\frac{I}{e} = C.$$

EXAMPLE. The moment of inertia of a 15-inch beam 50 pounds per foot = 522.6. Distance of extreme lamina, 7''.5.

$$\text{Coefficient} = 8000 \times \frac{522.6}{7.5} = 557{,}500.$$

PROPERTIES OF WROUGHT-IRON I BEAMS.

Depth of Beam.	Weight per ft.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia, axis perpendicular to web at centre.	Coefficient, 12,000 lbs. strain.
inches.	lbs.	inches.	inches.	inches.		
20	90.7	27.2	.69	6.75	1650.3	1,320,000
20	66.7	20.0	.50	6.00	1238.0	990,000
15	80.0	24.0	.76	6.08	813.7	868,000
15	66.7	20.02	.50	6.00	707.0	748,000
15	60.0	18.0	.57	5.45	625.5	667,200
15	50.0	15.0	.49	5.05	522.6	557,500
* 12¼ H.	56.7	16.77	.60	5.50	391.2	511,000
12	56.5	17.0	.78	5.16	348.5	464,800
12	42.0	12.6	.51	4.63	274.8	366,400
12¼ L.	41.7	12.33	.47	4.79	288.0	377,000
10½ H.	45.0	13.36	.47	5.00	233.7	356,000
10½	40.0	12.0	.55	4.80	201.7	307,200
10½ L.	35.0	10.44	.38	4.50	185.6	283,000
10½	31.5	9 5	.41	4.53	165.0	251,200
10½ Ex. L.	30.0	8.90	.31	4.50	164.0	250,000
10	42.0	12.6	.50	4.75	198.8	318,100
10	36.0	10.8	.44	4.50	170.6	273,000
10	30.0	9.0	.37	4.31	145.8	233,300
9	38.5	11.6	.46	4.71	150.1	266,900
9	28.5	9.6	.40	4.16	110.3	196,000
9	23.5	7.1	.34	3.96	92.3	164,000
8	34.0	10.2	.50	4.50	102.0	203,900
8	27.0	8.1	.41	4.09	82.5	165,100
8	21.5	6.5	.33	3.71	66.2	132,300
7	22.0	6.6	.38	3.82	51.9	118,500
7	18.0	5.4	.26	3.52	44.2	101,100
6	16.0	4.8	.25	3.44	29.0	77,400
6	13.5	4.1	.24	3.24	24.4	65,100
5	12.0	3.6	.28	2.96	14.4	46,000
5	10.0	3.0	.23	2.85	12.5	40,000
4	7.0	2.1	.18	2.50	5.7	22,800
4	6.0	1.8	.18	2.18	4.6	18,300
3	9.0	2.7	.40	2.58	3.5	18,900
3	5.5	1.7	.16	2.22	2.5	13,400

To find the safe load in pounds equally distributed, divide the coefficient by the span in feet. To find the safe load in pounds, weight in centre of span, divide the coefficient by the span in feet, and take one half the quotient.

Deflection.—To find the deflection of beams for the above distributed loads, divide the square of the span in feet by 70 times the depth of beam in inches.

*** Letters designate Heavy and Light sections.**

Coefficients for Steel Beams.—If L be given in feet, as before for iron beams, but using 16,000 pounds strain, then

$$WL = 10{,}666 \frac{I}{e} = C.$$

EXAMPLE. The moment of inertia of a 9-inch beam 27 pounds per yard is 110.6. Distance of extreme lamina, 4.5.

$$\text{Coefficient} = 10{,}666 \times \frac{110.6}{4.5} = 262{,}200.$$

PROPERTIES OF STEEL I BEAMS.

Depth of Beam.	Weight per ft.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia, axis perpendicular to web at centre.	Coefficient, 16,000 lbs. strain.
inches.	lbs.	inches.	inches.	inches.		
24	100	30.0	.75	7.20	2322.3	2,064,000
24	80	23.2	.50	6.95	2059.3	1,830,500
20	80	23.5	.60	7.00	1449.2	1,545,600
20	64	18.8	.50	6.25	1146.0	1,222,400
15	75	22.1	.67	6.31	757.7	1,077,300
15	60	17.6	.54	6.04	644.0	916,300
15	50	14.7	.45	5.75	529.7	753,300
15	41	12.0	.40	5.50	424.1	603,200
12	40	11.7	.39	5.50	281.3	500,100
12	32	9.4	.35	5.25	222.3	395,200
10	32	9.7	.37	5.00	161.3	344,000
10	25.5	7.5	.32	4.75	123.7	263,800
9	27	7.9	.31	4.75	110.6	262,200
9	21	6.2	.27	4.50	84.3	199,900
8	22	6.5	.27	4.50	71.9	191,600
8	18	5.3	.25	4.25	57.8	154,000
7	20	5.9	.27	4.25	49.7	151,400
7	15.5	4.6	.23	4.00	38.6	117,600
6	16	4.7	.26	3.63	28.6	101,800
6	13	3.8	.23	3.50	23.5	83,500
5	13	3.8	.26	3.13	15.7	67,000
5	10	3.0	.22	3.00	12.4	52,900
4	10	2.9	.24	2.75	7.7	41,200
4	7.5	2.0	.20	2.63	5.9	31,400

To find the safe load in pounds equally distributed, divide the coefficient by the span in feet. To find the safe load in pounds, with weight in centre of span, divide the coefficient by the span in feet, and take one half the quotient.

Channels—are placed against walls in place of I beams to receive the wall arches.

PROPERTIES OF WROUGHT-IRON CHANNELS.

Depth of Channel.	Weight per ft.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia, axis perpendicular to web.	Coefficient, 12,000 lbs. strain.
inches.	lbs.	inches.	inches.	inches.		
15	63.3	18.85	.75	4.75	586.0	625,000
15	60	18.00	.93	3.93	473.1	502,000
15	40	12.00	.50	4.00	376.0	401,000
12½	46.6	14.10	.68	4.00	291.6	381,000
12½	23.3	7.00	.33	3.00	153.2	201,100
12	50	15.00	.97	3.23	247.3	329,600
12	30	9.00	.47	2.73	175.3	233,600
12	20	6.00	.32	3.01	120.2	159,100
10½	20	6.00	.375	2.75	88.4	134,750
10	35	10.50	.75	2.95	126.3	202,400
10	20	6.00	.30	2.50	88.8	142,400
10	16	4.80	.32	2.51	62.8	100,800
9	23.3	7.02	.43	3.125	82.1	146,000
9	30	9.00	.71	2.83	87.8	156,800
9	18	5.40	.31	2.43	63.5	113,600
9	16.6	5.08	.33	2.5	58.8	104,000
8	28	8.40	.76	2.80	63.9	128,000
8	15	4.48	.26	2.5	44.5	88,950
8	11	3.30	.20	2.2	32.9	65,800
7	20	8.40	.76	2.8	63.9	128,000
7	12	3.60	.25	2.5	27.1	62,000
6	16	4.80	.52	2.34	22.3	59,600
6	11	3.20	.28	2.25	17.2	45,700
5	14	4.20	.56	2.24	13.10	41,900
5	6	1.80	.15	1.65	7.16	22,900
4	9	2.70	.39	1.89	5.75	23,100
4	5	1.50	.17	1.49	3.69	14,800
3	6	1.80	.33	1.65	2.22	11,800
3	5	1.45	.20	1.50	2.0	10,500

To find the safe load equally distributed, divide the coefficient by the span in feet. For a safe centre load take one half the quotient.

NOTE.—Inasmuch as there is a great diversity in published tables of safe load for beams, etc., every one must judge for himself what proportion of the elastic strength of the beam will best suit his purpose.

STEEL CHANNELS.

Depth of Channel.	Weight per ft.	Area of Section.	Thickness of Web.	Width of Flange.	Moment of Inertia, axis perpendicular to web.	Coefficient, 16,000 lbs. strain.
inches.	lbs.	inches.	inches.	inches.		
15	32.00	9.4	.40	3.40	284.5	404,700
15	51.00	15.0	.775	3.76	390.0	554,700
12	20.00	5.9	.30	2.90	117.9	209,600
12	30.25	8.9	.55	3.15	153.9	273,600
10	15.25	4.5	.26	2.66	63.8	136,100
10	23.75	7.0	.51	2.91	84.6	180,500
9	12.75	3.7	.24	2.44	43.3	102,700
9	20.50	6.0	.49	2.69	58.5	138,700
8	10.50	3.0	.22	2.22	28.2	75,300
8	17.25	5.0	.47	2.47	38.9	103.700
7	8.50	2.5	.20	2.00	17.4	53.100
7	14.50	4.3	.45	2.25	24.6	75,000
6	7 00	2.1	.19	1.89	11.1	39,400
6	12.00	3.6	.44	2.14	15.6	55,400
5	6.00	1.7	.18	1.78	6.5	27,900
5	10.25	3.0	.43	2.03	9.1	39,000
4	5.00	1.4	.17	1.67	3.5	18,700
4	8.25	2.4	.42	1.92	4.8	25,700

To find the safe load equally distributed, divide the coefficient by the span in feet. For a safe centre load take one half the quotient.

Limits for the Safe Load.—The previous tables of coefficients are to be used for the greatest safe loads, and the beams are entirely reliable for them under ordinary conditions.

The character of the load must be considered, and the mode of application.

The loads may be suddenly applied, and the beams subject to vibration, or they may be of considerable length without lateral support.

In many such cases it may be necessary to take smaller loads.

Then again, if the beams are "*rigidly secured*," as in Fig. 40, with heavy knees and a number of bolts, a proportionately larger load could be applied.

The following limitations will be proper for specified conditions:

Character of Loading.	Greatest Safe Load.
Quiescent loads, subject to little vibration, as in ordinary floors, etc., especially where beams are short,	As in tables.
Fluctuating loads, causing vibration, especially if the beams are long as compared to their depth.	One fifth ($\frac{1}{5}$) less than tables.
When loads are suddenly applied, or exposed to vibration from machinery or rapidly moving loads.	One third ($\frac{1}{3}$) less than tables.

If beams are supported as described below, the greatest safe loads will bear the given ratios to the above tables:

Character of Beam.	Greatest Safe Load.
Fixed at one end, with the load concentrated at the other end.	One eighth ($\frac{1}{8}$) part of that found by tables.
Fixed at one end, with the load uniformly distributed.	One fourth ($\frac{1}{4}$) of the tables.
Rigidly fixed at both ends, with the load in the middle of beam.	Same as found by tables.
Rigidly fixed at both ends, with the load uniformly distributed.	One and one half ($1\frac{1}{2}$) times that found by the tables.
Continuous beam, loaded in middle.	Same as found by the tables.
Continuous beam, load uniformly distributed.	One and one half ($1\frac{1}{2}$) times that found by the tables.

Beams with Fixed Ends.—By beams "rigidly secured," as denoted by the above, is meant that the beam must be securely fastened at both ends by being connected by knees, or so firmly secured that the connection will not be severed if the beam was exposed to the ultimate load.

In this case the beam is of the same character as if continuous over several supports, or as if consisting of two cantilevers, the space between whose ends are spanned by a separate beam.

Beam Connections.—The beams and their spacing having been determined upon, the next but not least important ques-

tion is that the connections are sufficiently strong to support the beams and their loads.

This is determined by riveting or bolting to the ends of each beam, when they connect with each other or to the girders, angle-knees with sufficient rivets or bolts to resist the shearing strain. The shearing strain on rivet or bolt is measured on the area of the cross-section. See examples on the following pages.

The following size angles can be economically used for all the different size steel beams in use, providing the end of beam does not rest upon a seat riveted to the girder. Two angles are used to each end.

Size of Beam.	Minimum Safe Span in Feet.	Size of Angles in Inches.	No. of Rivets, ¾" diam., in each Leg.
20" × 84 lbs. per ft.	17' 0"	4" × 4" × ⅜" × 15" long	5
20 × 64 " "	16 0	" " " "	5
15 × 75 " "	12 0	6" × 6" × 7/16" × 10" "	5
15 × 60 " "	11 5	" " " "	5
15 × 50 " "	11 0	" " " "	5
15 × 41 " "	10 5	" " " "	5
12 × 40 " "	8 5	6" × 6" × 7/16" × 8" "	5
12 × 32 " "	7 5	" " " "	5
10 × 33 " "	10 5	6" × 6" × 7/16" × 6½" "	3
10 × 25.5 " "	9 0	" " " "	3
9 × 27 " "	9 5	6" × 6" × 7/16" × 5" "	3
9 × 21 " "	8 0	" " " "	3
8 × 22 " "	8 0	" " " "	3
8 × 18 " "	7 0	" " " "	3
7 × 20 " "	6 0	6" × 6" × ⅜" × 5" "	3
7 × 15.5 " "	5 5	" " " "	3
6 × 16 " "	6 5	3½" × 3½" × ⅜" × 2¾" "	1
6 × 13 " "	6 0	" " " "	1

To get the full strength of the beams: if the spans are increased the number of rivets would decrease, and *vice versa.*

New York Building Law Relating to Beam Connections.—" SEC. 484. All iron or steel trimmer-beams, headers, and tail-beams shall be suitably framed and connected together, and the iron girders, columns, beams, trusses, and all other ironwork of all floors and roofs shall be strapped, bolted, anchored,

and connected together and to the walls in a strong and substantial manner. Where beams are framed into headers, the angles which are bolted to the tail-beams shall have at least two bolts for all beams over seven inches in depth, and three bolts for all beams 12 inches and over in depth, and these bolts shall not be less than $\frac{3}{4}$ inch in diameter.

"Each one of such angles or knees when bolted to girders shall have the same number of bolts as stated for the other leg.

"The angle-iron in no case shall be less in thickness than the header or trimmer to which it is bolted, and the width of angle in no cases shall be less than one third the depth of beam, excepting that no angle-knee shall be less than $2\frac{1}{2}$ inches wide, nor required to be more than 6 inches wide.

"All wrought-iron or rolled-steel beams 8 *inches deep and under* shall have bearings *equal to their depth* if resting on a wall; 9 *to* 12-*inch beams shall have a bearing of* 10 *inches*, and all beams more than 12 inches in depth shall have bearings of not less than 12 inches if resting on a wall.

"Where beams rest on iron supports and are properly tied to the same, no greater bearings shall be required than $\frac{1}{8}$ the depth of the beams.

"Iron or steel floor beams shall be so arranged as to spacing and length of beams that the load to be supported by them, together with the weight of the materials used in the construction of the said floors, shall not cause a deflection of the said beams of more than $\frac{1}{30}$ of an inch per lineal foot of span; and they shall be tied together at intervals of not more than 8 times the depth of the beam."

In comparing the strength of the beams with the number of bolts required, we will be able to determine at what span the above law would apply. We will take the smallest number of bolts and the largest beam allowed for that number, which will be a 10 × 32 lbs. per foot I.

We first determine the value of the $\frac{3}{4}$-inch bolts in each end of the beam, being four in number, two for each end.

The greatest safe shear allowed upon bolts and rivets by the New York Building Law is 9000 lbs. per square inch.

The area of a $\frac{3}{4}$-inch bolt $= .4418$ square inches; this multiplied by $9000 = 3972$ lbs., the safe shear each bolt would sustain in single shear, being in double shear twice the value, or $2 \times 3976 = 7952$ pounds. Multiplying this by the whole number of bolts in each end equal $7952 \times 4 = 31{,}808$ lbs., the entire strength of the bolts in the beam.

By referring to the table of steel beams the coefficient of a 10×32 lbs. per foot I $= 344{,}000$; this divided by $31{,}808 = 10.8$, the least span in feet to which the beam could be used to make the strength of beam and connection equal.

If the above beam is used over a larger span the connection will require less bolts and *vice versa.*

Suppose we now try the $15'' \times 60$ lbs. per foot I, in which 3 bolts are required, and determine the span we could use the number of bolts designated.

The safe shearing value of the bolts, 3 in each end, would be 6 bolts $\times$ $7952 = 47{,}712$ lbs., the entire strength of the bolts.

Then, by the same table, the coefficient of the beam is 916,300; this divided by $\frac{916.300}{47{,}712} = 18.1$ ft., the least span in which the beam could be used to make the strength of beam and connection equal.

The legs of the angles against the header, girder, or beam to which the framed beam is secured should have twice the number of bolts; that is, the bolts in these legs are in single shear, they therefore require the above number.

Of the different beams and their connections used in the floor framing, those shown in Fig. 40*a* seem to cover this number; 20-inch beams being used mostly for girders, they are not shown in the figure. The 15-inch beams have 6 bolts $\frac{3}{4}$ inch in diameter at each end, or equal to 12 in the entire beam; then $12 \times 7952 = 95{,}424$ lbs. The coefficient 916,300 divided

by 95,424 = 9.6 feet. The 12-inch beams, 40 lbs. per foot, **have 10** bolts in all; then 10 × 7952 = 79,520 lbs. The coef-

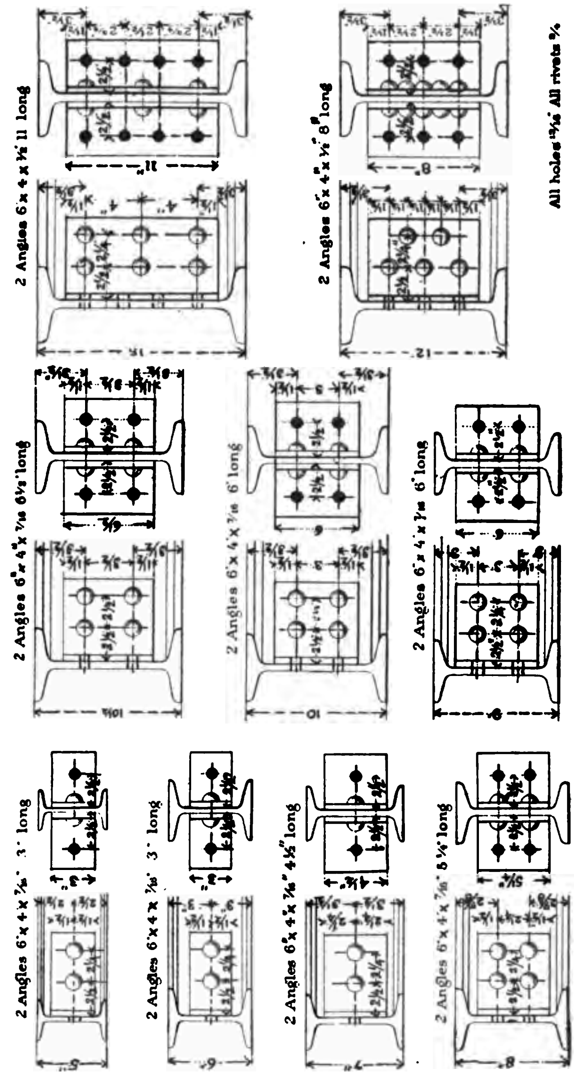


FIG. 40*a*.

ficient 500,100 divided by 79,520 = 6.2 feet. Continue in like manner with the other sizes.

Floor Arches.—The common mode of filling-in between the beams of a floor was the brick arch; but this is largely out of use, and in its place a cheap material is now extensively used, which consists principally of burnt fire-clay, and among the many qualities possessed by the *hollow fire-clay blocks* is their special application to fire-proof floors.

1. They are absolutely fire-proof, having been submitted during their course of manufacture to a white heat.

2. They are water-proof, and can be erected as fast as the walls will admit of the beams being set. In case of an incipient fire, water poured on the floors can have no bad effect to their solidity.

3. They offer a flat surface on the bottom and top after they are laid, thereby giving a flat ceiling for plastering and a flat surface for floor-sleepers and filling.

4. They are much lighter than the old solid brick arch, and are free from shrinkage.

5. They can be made any depth and accommodated to large and small spans.

WEIGHT AND SAFE SPANS FOR HOLLOW FIRE BLOCKS.

Width of Span.	Depth of Arch.	Weight per Sq. Ft.	Safe Load in Lbs. per Sq. Ft.
3 ft. 6 in. to 4 ft.	6 in.	29 lbs.	1,000
4 ft. to 4 ft. 6 in....	7 "	33 "	1,200
4 ft. 6 in. to 5 ft.	8 "	37 "	1,400
5 ft. 6 in. to 6 ft.	9 "	40 "	1,500
6 ft. 0 in. to 6 ft. 6 in....	10 "	43 "	1,500
6 ft. 6 in. to 7 ft. 6 in....	12 "	48 "	1,800

The 6-inch block to be used for light purposes, 8-inch for office buildings, 10-inch for theatres, and the 12-inch for warehouses.

The manner of applying hollow blocks is shown in Fig. 41. Those adjoining the beam are called *skew backs*, and made with a shoulder formed to fit the flange, and extend $\frac{3}{4}$ to $1\frac{1}{2}$ inches below, completely covering the beam, as at Fig. 6. The centre

block is the key, and the intermediate blocks are so placed, when laid in cement, that they form a complete arch. All are dove-tailed to receive the plastering, as shown in Fig. 5.

Brick Arches.—Brick arches properly built between the beams are practically indestructible from any usage that would occur in a building.

That brick arches will endure considerably more than the beams would sustain was shown by the loading of an arched floor at the Watertown Arsenal, Mass.

" A floor 29 ft. square was composed of five 15-inch I-beams, 200 lbs. per yard, carrying brick arches.

" The beams were 7 ft. 4.8 in. apart on centres, and rested on brick walls 28 ft. 6 in. apart. The size of the brick arches was 8½ in. in the 7 ft. 4.8 in. span. Soft-burned bricks were used, laid on edge with lime mortar. The arches were backed or levelled on top with concrete and planked over. The maximum load carried by the floor was 563 lbs. per square foot, and the beams failed and not the arches. This load caused a continuous yielding of the beams, which was allowed to continue. All the floor was deflected a distance of 13.07 in., measured at the centre of the middle beams. The brick-work endured this great deflection, and apparently would have stood much more without failure.

Porous Terra-cotta Arches is a mixture of clay and saw-dust, or any other combustible matter may be substituted, such as shavings, tanbark, and charcoal. After the compound is properly mixed the blocks are moulded, and, when sufficiently dry, placed in a kiln prepared for the purpose and subjected to a great heat adequate to consume all the combustible matter, leaving the blocks porous.

For hanging ceilings, etc., these blocks are made in different sizes. The T's are spaced about 25 in. centres; then the blocks, 24 in. wide, are set in place with cement, as shown in Figs. 1 and 2.

Fig. 4 is a column protected by a casing of ribbed blocks

with an air space between. The blocks are dovetailed on the outside for holding the plastering and set with cement, and

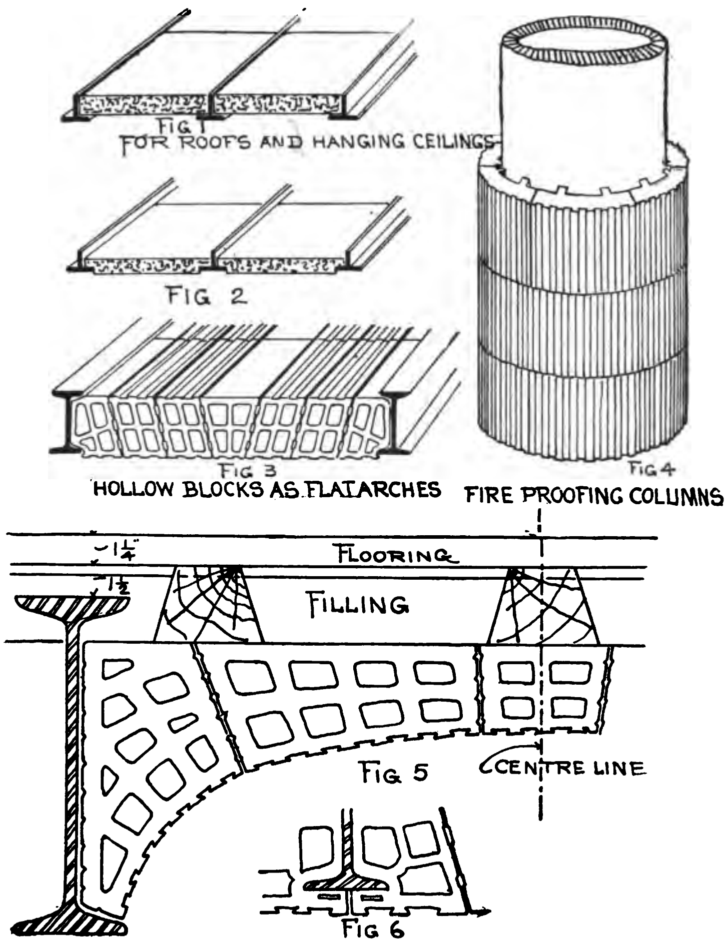


FIG. 41.

firmly secured with copper wire bound on the outside. Square cast iron, wrought, or steel columns are made fire-proof in various ways by the terra-cotta blocks.

WEIGHT OF POROUS TERRA-COTTA FOR FURRING, ROOFING, AND CEILING.

Hollow clay furring................	2 in. thick	12 lbs. per sq. ft.
Porous terra-cotta furring...........	2 "	8 " "
" " " roofing..........	2 "	12 " "
" " " "	3 "	16 " "
" " " ceiling...........	2 "	11 " "
" " " "	3 "	15 " "

WEIGHT OF HOLLOW BURNT CLAY AND POROUS TERRA-COTTA PARTITIONS.

Hollow Clay.		Porous Terra-cotta.	
3 in. thick....	14 lbs. per sq. ft.	3 in. thick....	12 lbs. per sq. ft.
4 "	18½ " "	4 "	17 " "
5 "	23 " "	5 "	23 " "
6 "	25 " "	6 "	27 " "
7 "	31 " "	7 "	31 " "
8 "	34 " "	8 "	36 " "

Concrete Arches.—Concrete arches are composed of broken stone, fragments of brick, pottery, and gravel, held together by being mixed with lime, cement, asphaltum, or other binding surfaces.

Mr. E. L. Ransome,* a successful worker of concrete in San Francisco, conceived the idea of using square bars of iron and steel, twisted the entire length, in place of flat bars and wires, as had been used by other experimenters on the subject. It was found that these bars were held in the concrete equally as well if not better than the others, and that they were much less expensive. The sizes used ranged from ¼ in. to 2 in. square.

A section of a flat floor in the California Academy of Science, 15 × 22 ft., was tested in 1890 with a uniform load of 415 lbs. per square foot, and the load left on for one month. The deflection at the centre of the 22-ft. space was only ⅛ in. It was estimated by the architects that the saving in this construction over the ordinary use of steel beams and hollow fire-

* Kidder's "Architects' Pocket Book."

blocks of the same strength, and with similar cement-finished floors on top, amounted to 50 cents per square foot of floor.

Corrugated Arches or Flooring.—The trough-shaped sections, known as Pencoyd corrugated flooring, as shown in Fig. 42, are now successfully used in the floors of buildings as well as bridges. The smaller section A is generally applied to buildings, B to bridges.

LOADS IN LBS. PER SQUARE FOOT WHICH CAUSE A DEFLECTION EQUAL TO $\frac{1}{400}$ OF THE SPAN, TABLE FOR "A" SECTION.

Weight of Material per Sq. Ft.		Span in Feet.										
Iron.	Steel.	5	6	7	8	9	10	11	12	13	14	15
14.5	14.8	2,460	1,400	900	600	420	300	230	180	140	110	90
18.0	18.4	3,000	1,750	1,100	740	520	380	290	220	170	140	110
21.5	21.9	3,600	2,120	1,300	900	630	460	340	250	210	170	130
25.0	25.5	4,200	2,500	1,570	1,050	740	540	400	310	240	200	160
28.5	29.1	4,800	2,850	1,800	1,200	850	620	460	360	280	220	180

Figs. 1 and 2 show the manner of supporting the flooring; Fig. 1 has wooden sleepers filled in between with ashes or concrete; this filling completely deadens the floor. In Fig. 2 the wooden flooring rests directly upon the iron-work.

It is very important to have at least 3 or 4 inches from the top of the girder beams to the finished floor, to allow some space between the wooden flooring and girders for the passing of pipes.

In Fig. 43 is another method of forming an arch between I-beams. At A, B is the depth of the corrugation; A is the width, which varies from 2 to 5 in.; B represents the application between the I-beams.

The Gustavino Tile Arch.—Within a few years a method of constructing floor arches by means of thin tile, cemented together so as to make one solid mass, has been introduced. The floors are constructed by covering the space between the girders by a single vault of tile 6″ × 8″ × ½″ thick, cemented together in three or more thicknesses, depending upon the size

of the arch. The thickness is generally increased at the haunches. The strength of these floors, considering their thickness, appears remarkable.

The system is employed in a number of buildings in New York and Boston, and seems to be very desirable among

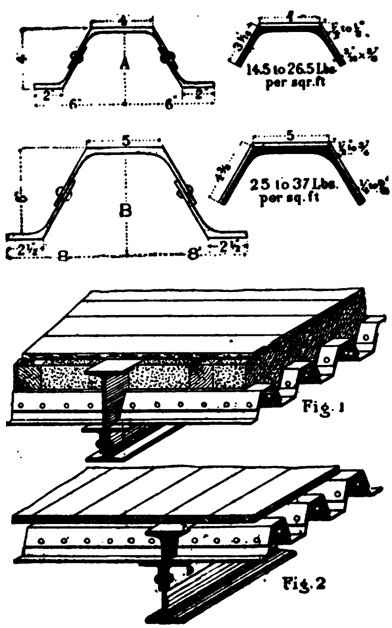


FIG. 42.

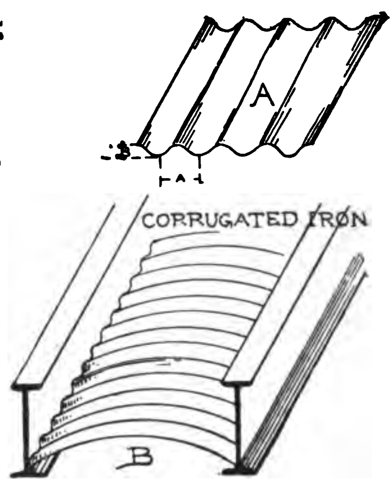


FIG. 43.

architects for vaulted ceilings for decoration purposes, as the vault can be made the size of the room.

Tie Rods.—The horizontal thrust of arches is provided for by the use of tie-rods from $\frac{5}{8}$ to $\frac{7}{8}$ inches in diameter; $\frac{3}{4}$ of an inch are more frequently used, spaced from 4 to 6 ft. apart in the centre of the depth of the beams. The thrust of brick arches per lineal foot can be found by the formula

$$T = \frac{1.5\,WL^2}{R},$$

in which W is equal to the load per square foot, $R =$ the rise of the arch in inches, and $L =$ the span in feet.

EXAMPLE.—The beams supporting an arched brick floor are 4 feet apart, and the rise of the arches is 4 inches.

The total dead load of floor and live load equal 150 lbs. per square foot. Then,

$$\frac{1.5 \times 150 \times 16}{4} = 900 \text{ lbs. pressure per lineal foot of arch.}$$

If $\frac{7}{8}$-inch diameter rods are used which have an effective section of .420 square inches, then $.42 \times 15{,}000$ (the greatest stress allowed on rods) $= 6300$ lbs., which is the greatest load the rod should be allowed to sustain, and $\frac{6300}{900} = 7$ feet $=$ greatest distance apart of the tie rods.

This same calculation can be applied to flat arches, taking one half of the depth of the arch for the rise; in an 8-inch arch 4 inches equal the rise.

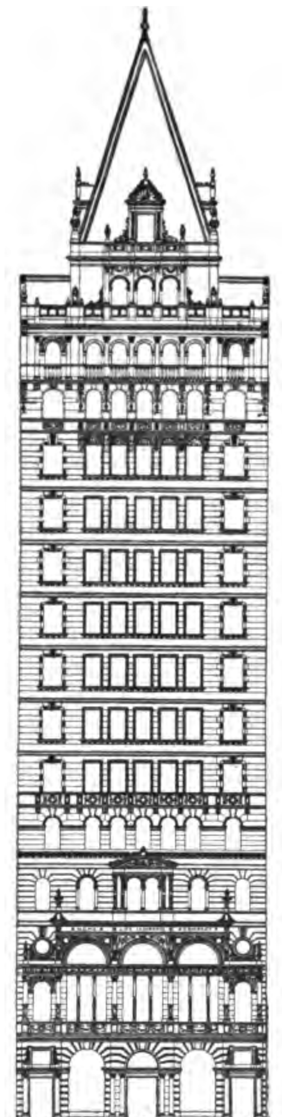


FIG. 45. — HOME LIFE INSURANCE BUILDING. FRONT ELEVATION.

CHAPTER V.

THE HOME LIFE INSURANCE BUILDING, N. Y.

THE new building for the Home Life Insurance Company, as designed by N. Le Brun & Sons, Architects, is situated at Nos. 256 and 258 Broadway, opposite the City Hall Park, on a plot of ground 55 feet 6 in. by 109 feet.

The construction is entirely fireproof and of the composite description, as provided for in the New York Building Law, passed April, 1892.

The constructive metal work is of wrought steel throughout, and with rare exceptions the various members are riveted together. The connections are designed with particular reference to the lateral stresses incidental to such a tall and narrow structure. As far as possible the constructive metal work is protected by terra-cotta or other fireproof material.

The front is built of white Tuckahoe marble, and is designed in the early Italian Renaissance style; elaborately decorated at the base with delicate carving, of simple finish in the middle portion, while it terminates

at the top with a boldly broken and picturesque outline. The height from the curb to the top of the marble pinnacle over the centre is 214 feet, while at the top of the spire it is 257 feet.

The marble of the front extends through the full thickness of the wall from base to cornice.

Below the sidewalk there are two stories, in the lowermost of which is situated all the machinery of the building for running the three elevators, the dynamos, the fans for ventilation, etc.

The company occupies several floors, and have their main office in the second story. The balance of the building is designed for renting. The same substantial character which distinguishes the constructive portions of the building has been continued in the balance of the work.

The stairs and elevator screens are of cast-iron, wrought-iron, and marble, designed in the same period of renaissance as the front. Much of this ornamental iron-work is electro-plated. The finish of the office floors are comparatively plain, but the office of the company and the main entrance hall are profusely decorated with relief ornaments.

The building was at first constructed upon a plot 30 feet 6 inches wide by 107 feet deep (see Figs. 47, 48, and 51); seven tiers of beams were set when it was decided to increase the size of the building by the purchase of the adjoining property, while another story was added to the original height (compare elevation, Fig. 45, with the transverse section Fig. 51 and the longitudinal section, Fig. 52).

The wall between the row of columns on the right was dispensed with and two columns placed each side of the centre, as shown on the beam plan, Fig. 46, in place of the right *F* column, which was transferred to the same position to the right of the increased lot as it previously occupied in the smaller lot.

A duplicate of the original plan (Fig. 47) was adopted in

the added area, with the exception that a large light court is placed in the centre, directly to the right or back of the elevators and shown on the beam plan, Fig. 46.

Beam Plan.—In arranging the beams upon the plan, economy of material has been strictly observed. The calculated weight, including dead and live load, is 175 lbs. per square foot of surface throughout the different floors, which includes a certain percentage for partitions, plastering, floor blocks, wooden flooring, tiling, etc.

The steel beams are spaced about 5 feet from centre to centre.

The three spans between columns marked *F* and *B* and the span between *C* and *R* are 9 inch by 70 lbs. and 9 inch 85 lbs. per yard.

The span between *B* and *C* are 12″ × 125 lbs. per yard, while the largest span between *B* columns have 15 × 123 lbs. per yard beams, and 9″ beams rest upon the top at right angles to the same and support the cast-iron mullions of the left light-court.

The double beam girders supporting the curtain-walls between columns *F* and *B*, *C* and *R* are 10″ × 90 lbs. per yard; between columns *B* a lattice girder; between columns *B* and *C*, 12″ × 125 lbs. per yard.

The *F* and *A*, *A* and *A* columns are spaced 13′ centres; *A* and *B*, 13′ 9″; *B* and *B*, 22′; *B* and *C*, 19′ 4″; *C* and *F*, 15′ $\frac{3}{8}$″.

In connecting the columns to each other and to the floor girders, knee-braces of plates and angles are extensively used. These braces are placed in the north and south wall column and made 3 and 4 feet in length, and reduced from that length to 18 inches in the top story (see section, Fig. 50).

Floor Plan.—The plan as shown at Fig. 47 describes in itself the area of the building as originally designed, divided to the best advantage into offices, halls, stairways, elevators, and light-courts; and, as previously mentioned, almost a duplicate of this plan was arranged in the additional 25 feet of

width. The wall at the right of building shown on the same plan was omitted.

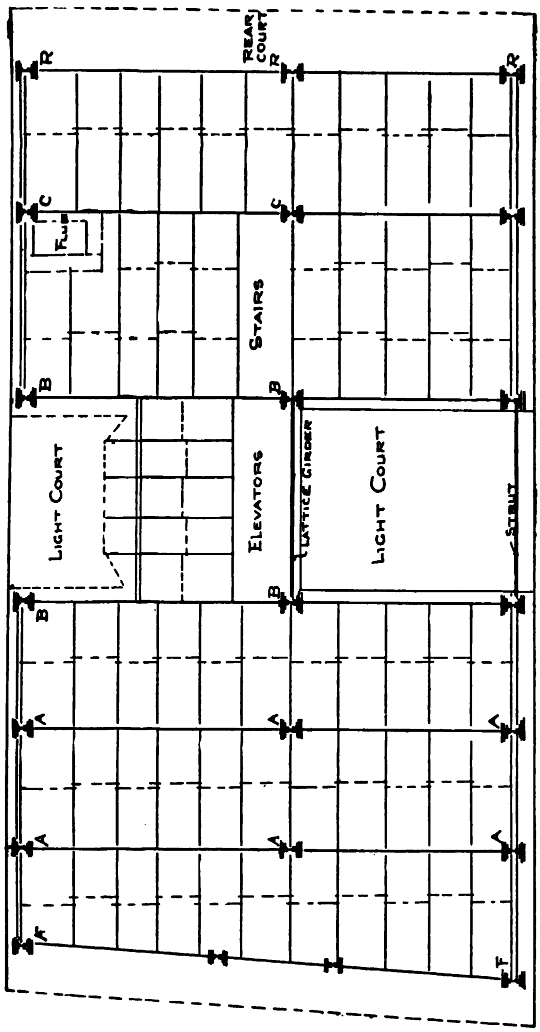


FIG. 46.—BEAM PLAN OF UPPER STORIES.

The front offices face Broadway; the inside offices are light through the light-courts, which extend from the lower floors to

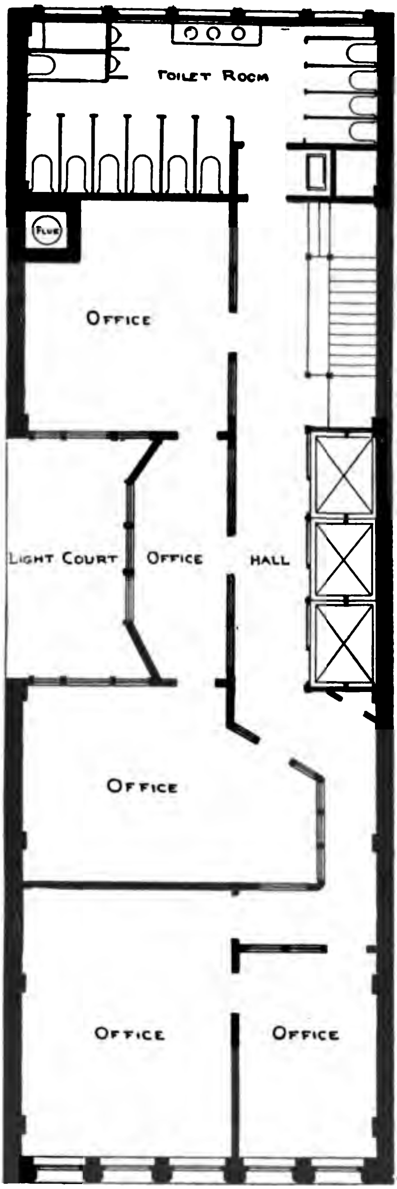


FIG. 47.—FLOOR PLAN AS ORIGINALLY STARTED.

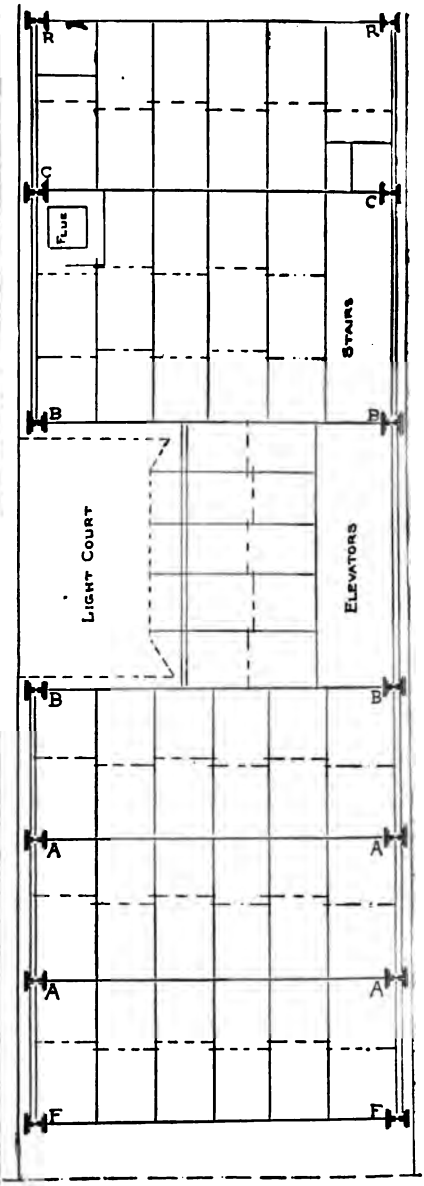


FIG. 48.—BEAM PLAN AS ORIGINALLY STARTED.

the roof. The walls enclosing the courts are built in between cast-iron mullions with flat moulded cast-iron cornices, and faced with white enamelled brick, the entire thickness being but 12 inches on all stories. The adjoining building on the left or south side has a corresponding light court, and the combined areas admit of so much light that these inner offices are equally as well light as those facing the street.

This plan shows a toilet-room in the rear of the building, but occurs only on a few stories; in the majority of the floors this space is utilized as an office.

The hall is also well-light as can be readily perceived, the partitions enclosing the same being punctured by windows to a considerable extent, and further light by a roof skylight and the large light court at rear of elevators.

Curtain Walls.—The side walls of the lower stories are as follows: Cellar, 2 ft. 4 in. thick; basement, 2 ft. 4 in.; first story, 24 in.; second story, 24 in.; third to sixth, 20 in.; sixth to tenth, 16 in.; the stories above the tenth are enclosed with a 12-in. wall, supported upon double beam girders, as previously described. The columns of the south wall are encased on the outside by 4 in. of this curtain wall, extending from the base to the roof. The north wall columns are placed 8 in. from the party wall, and faced on the inside by fireproof blocks, thus ensuring a complete covering in case of fire. The rear wall of building as well as the light-courts are shown in part elevation, plan and section at Fig. 49. This wall is 12 in. thick, built in as shown, supported its entire width by the plate girders between the *R* columns, as shown in the section. The cornice above the windows is first set in place; then the wall is built, and the cast-iron window-sill set. The mullions on the side nearest the columns are built in with the brick-work; the inside mullions are secured top and bottom to the bottom and top of plate girder. The entire combination is so constructed as to give to the outside an appearance of solidity, yet it takes but little area from the lot.

This figure also shows the connection of the *R* columns with the girder in such a manner that the column joint is perfectly rigid without the use of the *knee-braces*, by referring to the elevation of column, it will be seen that the joint is in the

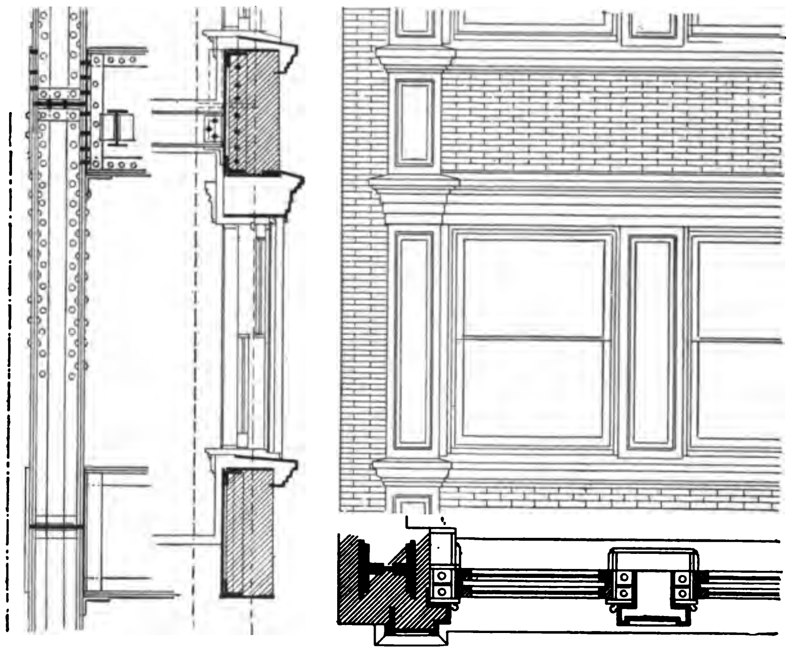


FIG. 49.—SECTION AND PART ELEVATION OF REAR AND NORTH COURT WALLS SHOWING MULLIONS AND FASCIAS.

centre of the girder; the girder being 4 feet high, the columns are practically stronger at the joint than at the body.

Columns and Girders.—The materials for the columns and girders for this building are fully described in the tables, with the height of stories and loads to be supported, and need but little description here; but it will be noticed that but few columns are required in the height of the entire structure (see Transverse Section, Fig. 51). This is not only an advantage in setting, etc., but also gives great rigidity to the frame, so that the danger of any movement of tops of columns is reduced to a minimum.

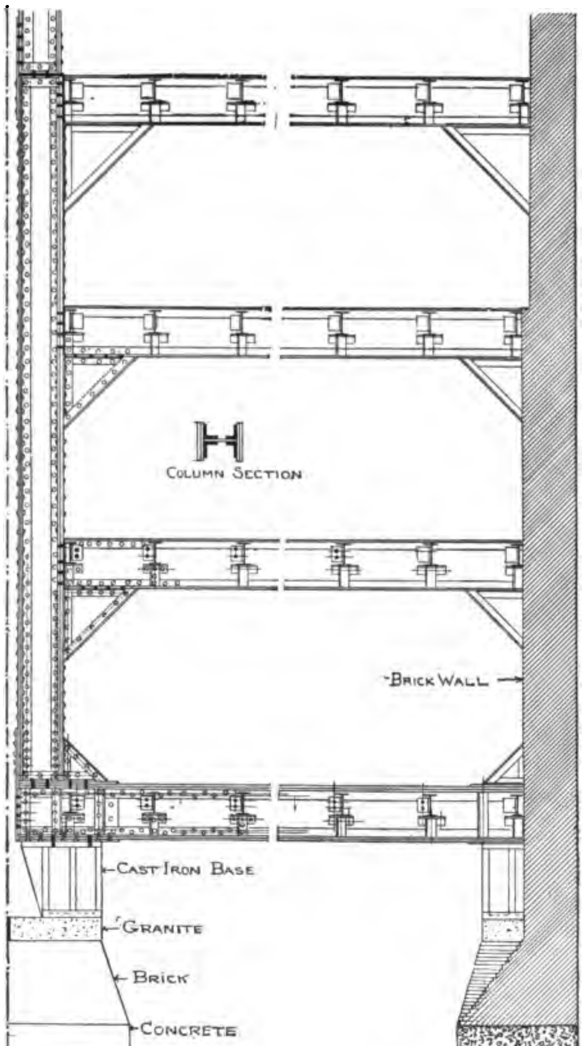


Fig. 50.—Transverse Section of North Wall Masonry, Elevation of South Wall Columns and Floor Girders.

STEEL COLUMNS HOME LIFE BUILDING.

Columns Marked "A."

	Length.		Outside Plates.		Webs.		Angles.		Load.
	Feet.	In.	No.	Size in In.	No.	Size in In.	No.	Size in In.	Tons.
Basement	10	0	4	$20 \times \frac{3}{4}$	1	$12 \times \frac{3}{4}$	4	$6 \times 4 \times \frac{3}{4}$	503
1st story	19	9	4	"	1	"	4	"	460.00
2d "	25	6	4	"	1	"	4	"	415.28
3d "	13	6	4	"	1	"	4	"	385.92
4th "	12	9	2 2	$20 \times \frac{5}{8}$ $20 \times \frac{1}{2}$	1 1	 "	4 4	" "	357.63
5th "	12	3	2 2	$16 \times \frac{3}{4}$ $16 \times \frac{1}{2}$	1 1	 $12 \times \frac{5}{8}$	4 4	" "	318.82
6th "	12	5	2	$16 \times \frac{3}{4}$	1	"	4	"	277.61
7th "	11	6	2 2	$14 \times \frac{9}{16}$ $14 \times \frac{1}{2}$	1 1	 "	4 4	" "	239.48
8th "	11	6	2	"	1	$10 \times \frac{5}{8}$	4	$5 \times 3\frac{1}{2} \times \frac{1}{2}$	202.35
9th "	11	6	2	$14 \times \frac{9}{16}$	1	"	4	"	161.52
10th "	11	6	2	$12 \times \frac{1}{2}$	1	$8 \times \frac{1}{2}$	4	"	134.60
11th "	11	6	2	"	1	"	4	"	104.00
12th "	11	6	2	"	1	"	4	"	74.38
13th "	13	0	2	"	1	"	4	"	45.87
14th "	11	9	2	"	1	"	4	"	17

Columns Marked "B."

Basement	10	0	2 4 2	$20 \times \frac{5}{8}$ $20 \times \frac{11}{16}$ $20 \times \frac{3}{4}$	1	$12 \times \frac{3}{4}$	4	$6 \times 4 \times \frac{3}{4}$	744.50
1st story	19	9	2	$20 \times \frac{3}{4}$	1	"	4	"	682.50
2d "	25	0	2 6	$20 \times \frac{5}{8}$ $20 \times \frac{5}{8}$	1	"	4	"	617.50
3d "	13	0	6	$20 \times \frac{5}{8}$	1	"	4	"	579.00
4th "	12	9	6	"	1	"	4	"	532.00
5th "	12	3	6	"	1	"	4	"	477.00
6th "	12	5	4	"	1	"	4	"	413.50
7th "	11	6	4 4	$16 \times \frac{5}{8}$ $16 \times \frac{1}{2}$	1	$19 \times \frac{5}{8}$	4	$5 \times 3\frac{1}{2} \times \frac{1}{2}$	356.50
8th "	11	6	2 4	$16 \times \frac{5}{8}$ $16 \times \frac{1}{2}$	1	"	4	"	304.00
9th "	11	6	4	$16 \times \frac{1}{2}$	1	"	4	"	244.00
10th "	11	6	4	"	1	$8 \times \frac{1}{2}$	4	"	201.00
11th "	11	6	4	"	1	"	4	"	155.00
12th "	11	6	2	"	1	"	4	"	111.00
13th "	13	0	2	"	1	"	4	"	68.25
14th "	11	9	2	"	1	"	4	"	30.30

STEEL COLUMNS HOME LIFE BUILDING.

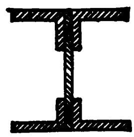
Columns Marked C.

	Length.		Outside Plates.		Webs.		Angles.		Load.
	Feet.	In.	No.	Size in In.	No.	Size in In.	No.	Size in Inches.	Tons.
Basement	10	0	4 2	$20 \times \frac{3}{4}$ $20 \times \frac{5}{8}$	1	$12 \times \frac{3}{4}$	4	$6 \times 4 \times \frac{3}{4}$	577.57
1st story	19	9	2	"	1	"	4	"	531. 0
2d "	25	0	4	$20 \times \frac{3}{4}$	1	"	4	"	493.20
3d "	13	0	4	"	1	"	4	"	450.00
4th "	12	9	2 4	$16 \times \frac{9}{16}$ $16 \times \frac{1}{2}$	1	"	4	"	418.50
5th "	12	3	4	"	1	"	4	"	368.50
6th "	12	5	4	$16 \times \frac{1}{2}$	1	"	4	"	326.00
7th "	11	6	2 2	$16 \times \frac{5}{8}$ $16 \times \frac{1}{2}$	1	$10 \times \frac{5}{8}$	4	$5 \times 3\frac{1}{2} \times \frac{1}{2}$	282.00
8th "	11	6	2	"	1	"	4	"	236.60
9th "	11	6	2	$16 \times \frac{5}{8}$	1	"	4	"	195.70
10th "	11	6	2	$14 \times \frac{9}{16}$	1	$8 \times \frac{1}{2}$	4	"	159.50
11th "	11	6	2	"	1	"	4	"	123.52
12th "	11	6	2	"	1	"	4	"	88.00
13th "	13	0	2	$14 \times \frac{1}{2}$	1	"	4	"	54.80
14th "	11	9	2	"	1	"	4	"	23.55

These columns were supplied with $\frac{3}{4}$ in. thick plates between, and at back of column, with knee-braces at the joints, and when connections were made into the transverse floor girders.

STEEL GIRDERS HOME LIFE BUILDING.

GIRDERS MARKED A.



2d tier.	Web 24″	× $\frac{3}{8}$″	× 26′–9$\frac{3}{4}$″	long,	6″ × 4″ × $\frac{7}{8}$ L's	
3d "	"	"	× "	"	"	
4th "	"	"	× "	"	"	
5th "	"	"	× 26–11$\frac{1}{4}$	"	"	
6th "	"	"	× 26–10$\frac{3}{4}$	"	"	
7th "	"	"	× "	"	"	
8th "	"	"	× 27–0$\frac{3}{4}$	"	"	
9th "	"	"	× 27–3$\frac{1}{2}$	"	"	
10th "	"	"	× "	"	6 × 4 × $\frac{3}{4}$	
11th "	"	"	× 27–5$\frac{1}{2}$	"	"	
12th "	"	"	× 27–9$\frac{3}{4}$	"	"	
13th "	"	"	× "	"	5 × 3$\frac{1}{2}$ × $\frac{5}{8}$	
14th "	"	"	× "	"	"	
15th "	"	"	× "	"	"	
16th "	"	"	× "	"	"	

Girders marked B, C, and F are similar to the above, but vary in length.

GIRDER MARKED R.

2d tier................	Web	24″ × $\frac{3}{8}$″ × 26′–11$\frac{3}{4}$″	4–6″ × 4″ × $\frac{7}{8}$″ L's.
3d to 16th tiers........	Web	48″ × $\frac{3}{8}$″ × 26′–11$\frac{3}{4}$″	2–3″ × 3″ × $\frac{3}{8}$″ L.
	Top plate	12″ × $\frac{5}{8}$″ × "	" "
	Bott. "	12″ × $\frac{5}{8}$″ × "	" "

FIRST TIER OF BOX GIRDERS.

GIRDERS MARKED "A."

4 top plates	20″ × $\frac{5}{8}$″ × 29′ 4$\frac{1}{2}$″	long.
4 bott. "	20″ × $\frac{3}{4}$″ × "	"
4 web "	28$\frac{1}{2}$″ × $\frac{1}{2}$″ × "	"
4 angles	6″ × 4″ × $\frac{5}{8}$″ × "	"

GIRDERS MARKED B.

4 top plates	20″ × $\frac{3}{4}$″ × 29′ 4$\frac{1}{2}$″	long.
2 " "	20″ × $\frac{5}{8}$″ × "	"
4 bott. "	20″ × $\frac{7}{8}$″ × "	"
2 " "	20″ × $\frac{3}{4}$″ × "	"
2 web "	26$\frac{1}{2}$″ × $\frac{5}{8}$″ × "	"
4 angles	6″ × 4″ × $\frac{7}{8}$″ × "	"

These girders were supplied with stiffeners, extra plates and cast separators in each end, to resist shearing and buckling of the webs.

The girders of the first tier rest upon cast-iron base plates, which set upon granite blocks, as shown in the section, Fig. 50.

Bases for the *A* columns are 4′ long, 3′ high, and $1\frac{1}{2}''$ thick; *B* bases, $7' \times 3' \times 2''$ and $1\frac{1}{2}''$ ribs; *C* bases, $5'\ 6'' \times 3' \times 1\frac{3}{4}''$; *F* and *R* bases, $3' \times 3' \times 1\frac{1}{4}''$. The columns do not set directly upon the bases, but rest upon the girders; the weight upon a column is transmitted through the girder to the base plate, the girder being stiffened to resist shearing and buckling of the webs by heavy wrought-steel plates, angle stiffeners, and heavy cast-iron separators, the centre line of pressure is thus extended farther from the party wall than it would be if the column was supported directly upon the granite block.

Specification of the several works, materials, matters and things to be doue and furnished for the cast and wrought iron and steel work, etc., for the building of the Home Life Insurance Company, at No. 256, 258 Broadway, in the city of New York.

All steel and iron work throughout the building is to be executed in the best and most substantial manner, and the contractor is to provide all requisite materials, implements, models, mechanical appliances, tools, carriage, scaffolding, etc., of every kind and description, necessary to properly execute and erect all the works, and protect the same during construction. The whole work is to be done according to the plans, drawings, and directions of N. Le Brun & Sons, architects, and subject to their approval.

General.—The transverse girders shown on framing plans are to be at right angles with the southern wall of building, with the exception of the first, against the Broadway front, and the beams will be parallel to the southern wall.

All the various members of the wrought constructional work are to be of steel. The general design ot the various parts is shown in the scale drawings and details, and the contractor is to include all the angles, splice plates, ties, braces, drilling, riveting, bolting, etc., etc., necessary to put the work

together in a perfect, workman-like manner, as may be hereafter directed or designed.

The columns are to be made in lengths of not over three stories.

The angles and plates in columns and girders to be whole from end to end, unless otherwise shown, or allowed by the architects.

The rivets for columns and girders to be $\frac{7}{8}$ inches in diameter, unless otherwise indicated.

The base plates under first tier of girders are to be set perfectly true and level.

The columns must be set so that their axes are perfectly plumb, and the outside faces of columns are to be kept plumb for full height of building, 4 inches from the outside face of walls. The columns in top story vary in height.

The connection between the columns, and connections between columns and girders are shown in general on the detail sheets. The connection between the girders and floor beams is to be by standard connections, with angle iron rests below the beams.

Each tier of beams is to be tied where shown in plans by $\frac{3}{4}$-inch rods, with heads and nuts at each beam, all of which is to be thoroughly tightened before the arching is built in between them.

The girders of first tier on which the columns rest are to be bolted or riveted to the cast-iron base, with separators accurately fitting plates, and are to be absolutely true on top and bottom surfaces, so as to give an even bearing throughout.

Quality of Steel.—All steel used is to be Bessemer or Open-Hearth steel. The tensile strength, limit of elasticity, and ductility shall be determined from a standard test piece cut from the finished material, and planed or turned parallel; the piece to have as near $\frac{1}{2}$ square inch sectional area as possible, and elongation to be measured on an original

length of 8 inches; two test pieces to be taken from each heat of rolled finished material, one for tension and one for bending. The tests are to be made in the presence of a representative for the owners, should such be deemed necessary, and all facilities for the purpose shall be afforded by the manufacturer.

Finished bars must be free from injurious seams, flaws, or cracks, and have a workmanlike finish, and shall have an ultimate strength of from 58,000 to 60,000 lbs. per square inch; elastic limit ½ the ultimate strength; minimum elongation 20 per cent in 8 inches; minimum reduction of area of fracture 40 per cent. This grade of steel is to bend cold 180 degrees to a diameter equal to thickness of the piece tested without crack or flaw on the outside of the bent portion.

Rivet Steel.—Rivet steel shall have a specified tensile strength of 60,000 lbs. per square inch, and is to be capable of bending double flat without sign of fracture on the convex surface of the bend.

Workmanship.—Inspection of the work shall be made at the mill by the owner's representative if deemed necessary, as the work progresses, and he shall have the right to condemn the whole or any part of the work, if in his opinion it is not to the standard required by the drawings and these specifications.

All workmanship must be first class. The rivet-holes for splice plates for abutting members shall be accurately placed, so that when members are brought into position

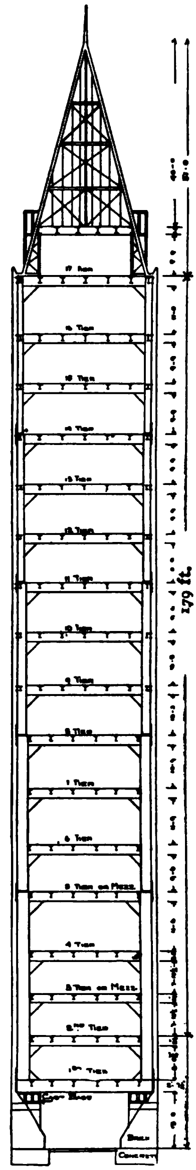


FIG. 51. — TRANSVERSE SECTION OF BUILDING ORIGINALLY STARTED.

the holes shall be truly opposite before the rivets are driven.

Rivets must completely fill the holes and have full heads concentric with the rivets of a height not less than 0'.6", the diameter of the rivet, and in full contact with the surface, or to be countersunk when so required, and machine-driven whereever practicable.

All abutting surfaces of compression members must be planed or turned to an even bearing, so that they shall be in perfect contact throughout.

Separators must accurately fit.

Built members must, when finished, be true and free from twists, kinks, buckles, or open joints between the component pieces.

Framing of Top Story and Spire.—The top story over main roof and the spire and the side of the elevator bulkhead are to be constructed of tie and angle iron, as shown in general on drawings. All to be properly framed and riveted together and strongly wind-braced, and firmly secured down to the steel frame-work of the building. The work is to include all framing around windows and doors, and all requirements by other mechanics necessary for them to put up their work complete. The columns supporting the girders and beams of roof are to be of steel frame-work, vertical on one side and inclined to the pitch of the spire on the other.

Painting at Works.—The entire steel-work of every description, including all columns, beams, girders, separators, plates, tie-rods, etc., to be cleaned of all scales and dirt, and painted with one good coat of best red lead and linseed oil, before it is brought to the ground.

All surfaces inaccessible after assembling must be painted two coats before the parts are assembled.

Sundry Work—Anchors.—All anchors, bolts, clamps, straps, dowels, rests for brick-work, etc., required to connect the metal construction with masonry, must be furnished and

made properly, as required by contractor for masonry and stone-work, to suit their relative positions and be of such dimensions and sections as directed. Anchors connected with beams, framings, etc., are to be securely fastened, care being taken that there is no looseness between the parts anchored.

All anchors, etc., for stone-work to be well galvanized.

Dimensions figured on the drawings are to be followed in all cases in preference to scale measures, and drawings and details to a larger scale in preference to those at a smaller scale, and they must all be verified by contractor; and if any error in dimensions or omissions in detail be discovered, either on the drawings or in these specifications, or discrepancies between the figures and actual construction in the building, the contractor must immediately report the same to the architects for rectification, explanation, or direction, as he will not be allowed to take advantage of the same, but shall carry out the details as originally intended and required, or as will be essential to the proper construction and finish of the work; and any and all drawings of details of construction made by the contractor for iron and steel-work must be submitted to the architects for their examination and approval before being put into execution.

Painting at the Building.—After the steel-work is built in position and just previous to the building of walls and floor arches, or the permanent covering of any of the wrought steel-work, the same is to be again thoroughly painted with best red lead and pure linseed oil, well laid on.

The cast-iron, after being thoroughly cleaned, is to be painted with a good coat of pure linseed oil previous to delivery at the building, and after delivery and approval at the building and before being put into position, is to receive a coat of red-lead paint as specified above for steel-work.

All bolts, nuts, etc., or any iron or steel surfaces from which the paint may have been removed in putting the work in place, are to be painted as above two coats.

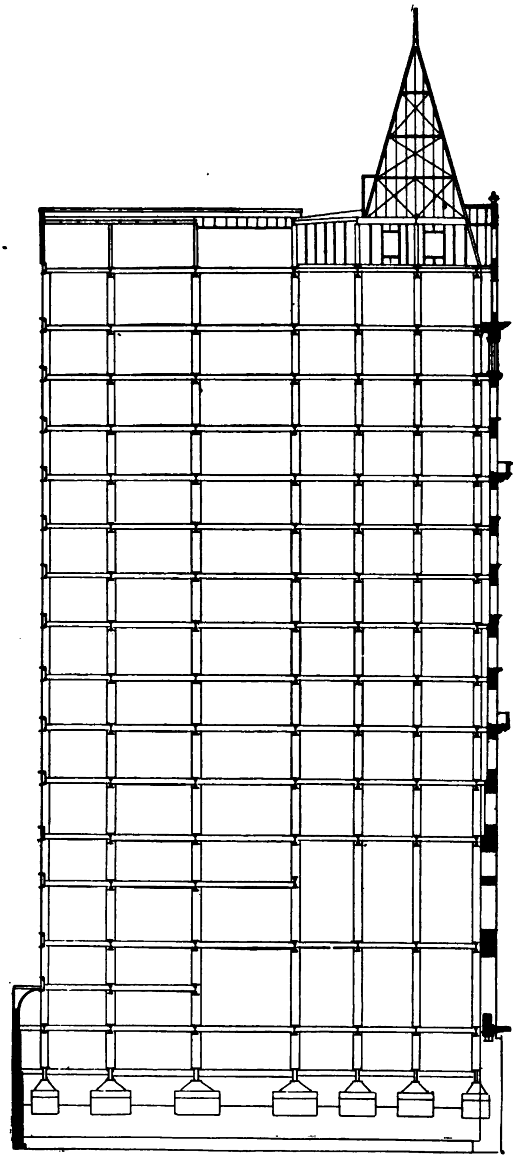


FIG. 52.—LONGITUDINAL SECTION OF BUILDING, AN ADDITIONAL STORY TO BE ADDED.

Cast-iron.—All cast-iron work to be of the best quality, and all castings must be sound, true, out of wind, clean, free from flaws, holes, or imperfections of every kind, and strictly made in accordance with the drawings and directions of the architects, to whom all patterns for the ornaments, mouldings, etc., must be submitted for approval before casting.

Lintels.—Cast-iron lintels are to be placed over the doors leading to the coal vaults.

Base for W. I. Smoke Flue.—The base for the wrought-iron smoke flue is to be of cast-iron, with flanges, bottom plate, etc., as shown on the drawings. The top flange is to be planed true, and is to be drilled for $\frac{3}{4}$-inch bolts to secure the bottom flange of the wrought-iron flue.

Plates.—Under wall ends of beams, cast-iron plates are to be furnished $8'' \times 12'' \times 1''$ to be set by the mason, but the contractor for iron-work in laying the beams is to verify the correctness of their position.

Door to Flue.—The cleaning-out door to the smoke-flue in cellar is to be of heavy cast-iron, strongly hinged to a cast-iron frame built in the masonry, and provided with approved fastening, etc., complete. The door is to fit absolutely tight.

Frame to Ash-lift.—The frame to ash-lift in sidewalk is to be of cast-iron, strongly made and to fit accurately to the soffit of the brick sidewalk arch. The upper flange, rebated in the pavement, is to be deeply corrugated or diamond roughened. The frame is to be made to receive a wrought-iron hinged grating hereafter specified.

Vault Lights.—Along the Broadway front, as shown on plan of first story, provide vault lights with extra heavy frame of cast-iron, rebated and provided with all necessary and approved supports, and stiffening webs, filled with concrete lights, and 2-inch diameter approved lenses. Beneath the window the lights are to be raised one step with ventilating riser, as will be detailed.

Similar lights are to be placed in the floor of basement over the entrance to boiler-room.

Curved Skylight.—Provide and place over the rear of first story, as shown on plans and longitudinal section, and extending across the whole width of building, a curved skylight made of heavy cast-iron frames, filled with best cement, light lenses 2½ inches in diameter. The same is to be finished with proper copper flashings and have cast-iron gutter at lower end of proper pitch, and outlet for leader connection and forming a wall-coping. Also provide with interior condensation gutters as will be directed and required.

All vault-lights and skylights, etc., must be made and guaranteed to be thoroughly water-tight, and be so maintained for two years after the completion of the building.

Coal-hole Covers were shown on plan. Furnish and set securely in pavement over coal vaults, two 24-inch round cast-iron coal-chute covers, filled with cement lenses, and to be provided with rebated frame, chain, fastening staples, etc., complete. The cover and frame are to be set flush with the sidewalk.

Sills to Doors on Roof.—Cast-iron sills, corrugated or diamond roughened, are to be placed to the doors leading from top floor to roof over rear, and from tank-room in spire to roof and to elevator bulkhead.

Bronze Saddles.—Furnish and put securely and neatly in position bronze saddles to the main entrance doors on Broadway, as will be approved.

Cast-iron Mullions, etc.; Rear and Courts.—Between the steel girders and columns forming the constructional part of the rear wall, and between the girders and beams of each story on light-courts are to be cast-iron columns or mullions, and moulded cast-iron sills and lintels or cornices, as shown on elevations. All are to be cast straight and smooth of full uniform thickness throughout, and to have all requisite lugs, brackets, connecting flanges, etc., cast on as directed, and

strictly conform to the detail drawings to be furnished, and be securely bolted to the girders, etc.

Columns to Elevator Shaft.—The columns to elevator shaft on various stories are to be of cast-iron, cast true and finished smooth in plain parts, and the carved and decorated parts to be sharp and hand-finished, as required. On first floor the columns are to be arranged to receive the marble pilasters shown on drawings. The necessary lugs to connect with floor and veiling beams are to be cast on. Models of the mouldings and carved ornaments are to be submitted to the architects for approval before casting.

Sills to Elevator Doors, etc.—The facias around elevator shaft at line of floors at each story are to be finished with cast and wrought-iron panels, to match the stair-work, and cast-iron sills and soffit mouldings at ceilings, all as detailed.

Stairs.—The stairs throughout the building are to be of iron construction, and finished in the most substantial, secure, and highly artistic manner, as per plans, sections, and full-sized drawings. All to be framed with substantial cast and wrought iron carriages, strings, braces, brackets, etc. The under-side and all exposed parts to be moulded, panelled, and thoroughly finished. The wall string to main stairs up to third story to be made "open" for marble base of wainscoting; above this story to top of building the strings are to be closed or panel-box strings. The outside string to be closed throughout. The risers to be solid and panelled. All to be properly lugged and flanged to receive the marble treads and platforms to be furnished by the contractor for marble-work, and the joints neatly and accurately fitted and bolted and fastened The facias around stair wells and the soffit mould are to be made to match the stairs. The rail to main stairs from first story to top story is to consist of a polished bronze or brass 2½-inch diameter rail. The rail to be supported on square newels and finished with carved bronze or brass plates at ends. The private stairs are to have similar rail 2 inches in diameter.

All requisite bends and casements to be carefully formed with easy sweeps and curves. The balusters or supports to hand rails are to be of decorated cast-iron in main stairs up to third story, artistically hand-finished, and the balance of the balustrade to main stairs and the whole of the private stairs to be of wrought-iron—all as shown on section and detailed. The spandrils to the first story and basement main stairs to be filled with cast-iron panel-work, smoothly finished and strongly and neatly put together.

The stairs from store to basement and from basement to cellar or engine-room to be of circular pattern with centre newel, cast-iron treads and platform corrugated or perforated, as may be directed, and open strings and risers and string wrought-iron railing, and hand-rail, and cast-iron newels, all of neat finish and substantial construction, as directed. The ladder to tank room in spire is to be made of wrought-iron with double run treads and wrought-iron bar hand rails. Similar ladder to be furnished from cellar to engineer's water-closet.

Do all drilling for marble setters and furnish the screws to secure the marble treads, etc., to the iron-work.

Guards to the Elevator Shaft.—The elevator shaft from the basement to the top story is to be enclosed with ornamental wrought-iron fret-work, as shown on the section, and as will be detailed. All is to be executed in the best artistic manner. The decorated portions of the first and second story screens to have hammered leaf and scroll-work. The doors throughout are to be wrought-iron, strongly framed, hung on approved anti-friction sheaves and steel-ways, and are to have approved large and strong latch fastenings. The elevator nearest stairs is to be a freight elevator, and the doors are to be arranged to open the full width of elevator car.

Electro-Plating.—All the iron-work about the main stairs and elevator guards from start to third story, including risers, strings, balusters, elevator posts, etc., complete, are to be

heavily electroplated in best manner, in brass, copper, or bronze, as will be more fully directed by the architects.

Partition to Cellar Stairs.—On basement story, the cellar stairs are to be enclosed by a solid wrought-iron screen from floor to ceiling. The screen is to be made of heavy crimped iron, secured to T-iron uprights and horizontal bars dividing it into panels. The door in this screen is also to be of wrought-iron, strongly hinged and secured with approved lock and spring catch. The door is to be made to fit absolutely tight. All to be done in the neatest and strongest manner, as will be approved.

Main Entrance Doors.—The doors to main entrance are to be hinged in three folds, to fold back against front pier, and are to be constructed of best bronze with panels and decorated mouldings. The doors are to be hung on strong bronze frame firmly secured to stone jambs, and are to be hinged in strongest manner and finished in the highest artistic style. Each fold to have top and bottom bronze bolts, and the doors are to be provided with best-made Yale & Towne bronze lock, etc., complete. All as will be approved.

Wrought-iron Boiler Flue.—The iron smoke stack, to be constructed in the brick flue as the building progresses, is to extend from the cast-iron shoe, specified above, in basement to eighteen inches above the top of chimney over roof, and is to be made of best wrought-iron $\frac{3}{8}$ inches thick, stiffened by $\frac{3}{8}'' \times 3''$ bands. The sections of which the flue is to be constructed are to be of such lengths as can be readily handled, and are in no case to exceed 20 feet.

The ends of such section are to have $3'' \times 3'' \times \frac{3}{8}''$ angles riveted on, through which the several sections will be tightly bolted or riveted together, and the joints between the sections are to be gas-tight, and all is to be put together as will be directed and approved. The stack is to be kept in a vertical position by wrought-iron bars built in the masonry flue, but care is to be taken in locating the bars, that they do not come

in contact with any joint or projecting stiffener, and that the stack has free play to expand.

Furring.—The panelled ceiling of entrance hall and the panelled ceiling and cornices of the main office are to be furred with angle irons $1\frac{1}{4}'' \times 1\frac{1}{4}''$, spaced about twelve inches between centres, approximately to the shape of the plaster mouldings, etc., all complete, as required by plasterer, for the attachment of the wire cloth or metal lathing. All furring is to be secured to the walls and suspended and secured to the beams, girders, and brackets in the most substantial manner, as approved.

Floors Beneath Elevators.—The floor beneath the two elevators stopping at first story and the floor beneath elevator stopping at basement are to be furred down two feet below the finished floors in a substantial manner with tee irons, secured to under side of beams and in walls. Floor over these tees with wrought-iron plates firmly secured to them, to make a substantial floor.

Skylight Over Main Office and Elevators.—Inclined flat skylights are to be provided over the main office ceiling beneath light-court and over the elevator shaft. The core of the bars to be made of *T*-shape iron of ample and approved strength, and must be enclosed by cold-rolled copper, 20 ounces to the square foot, to afford a support to the glass and form double condensation gutters. They are to be glazed with $\frac{1}{4}$ in. rough plate or ribbed glass laid in white-lead putty in best manner. Along the lower edge of skylight over main office, provide a substantial cast-iron gutter with proper pitch and outlet for leader connection, and to flash into the wall as shown and form a coping. Beneath this skylight is to be placed a strong wire screen, supported on strong wrought-iron frame made in movable panels securely put up, as will be approved.

Glass Under Skylight.—Beneath the above skylight in main office is to be a ceiling-light, of crackled and other glass, to be provided in a separate contract; but the contractor for iron-work is to furnish and put up the wrought-iron angle

frames to support the glass, as shown on ceiling plan. All to be done in the strongest manner, as approved.

Skylights in Top Story.—Over the stairs to top story is to be placed a best-make turret ventilating skylight of copper, with vertical sashes on sides and ends. The sashes are to be hung on pivots on two sides. All to be constructed in the most substantial manner. The skylight is to be constructed of hollow copper bars, provided with gutters, to connect with cross-gutters at upper and lower sides; the gutters at lower sides to be provided with indirect openings for the escape of condensed moisture, and at the same time be perfectly weather-tight. The sashes are to be provided with approved fastenings and apparatus for keeping same open at any angle. The glazing is to be done in the best manner with ¼-inch thick best quality ribbed plate glass of uniform tint.

All the above skylights must be made and guaranteed to be thoroughly weather-tight, and be so maintained for two years after the completion of the building.

Floor-lights.—Floor-lights are to be furnished where shown on plans of first and second stories. All to be made of heavy wrought-iron frames, strongly supported and filled with one-inch thick hammered 12-inch square glass, or as shown and directed, the glass to be bedded in white-lead putty.

Window Guards.—Window guards of round ¾-inch bars, set 4½ inch, on centres, are to be secured to the iron posts or mullions of rear in the first-story mezzanine and second-story. All to be framed out and put up in best secure manner.

Grating Doors Over Ash-lift.—On sidewalk over elevator, or lift for ashes, furnish and set complete a pair of wrought-iron grating doors, hung on the cast-iron flanged and rebated frame before specified, with strong hinges and provided with proper fastenings, and ratchet to keep open, and with safety-guards.

Platform Over Elevators.—A strong wrought-iron grat-

ing platform is to be built in the elevator shaft just below the machinery overhead.

Clamps.—Furnish the necessary clamps of wrought-iron to the carpenter to secure the wooden sleepers to the iron beams, as will be required.

Jobbing.—Do all cutting, drilling, and fitting of iron-work, and furnish and insert all requisite bolts, holdfasts, etc., required by other mechanics.

General.—The contractor is to remove from the premises all rubbish arising from his operations as the work proceeds and at completion of same. He must comply with all municipal or corporation ordinances and the laws and regulations relating to buildings in the city of New York. He will be liable and responsible for any damage to life, limb, or property that may arise or occur to any party whatever, either from accident or owing to his negligence or that of his employés during the operations of constructing or completing the works herein specified.

Should any difference of opinion or dispute arise between the contracting parties in relation to the true meaning of the plans, drawings, or these specifications, reference is to be made to the architects, whose decision on all such points shall be final and conclusive. No additional work will be allowed unless ordered by the architects in writing, and the order countersigned by the agent of the company; and no bill for work so ordered will be approved by the architects unless it is rendered immediately upon completion and approval of the said work.

No signs of any description will be allowed to be placed on or about the building or premises.

CHAPTER VI.

THE HAVEMEYER BUILDING.

THE Havemeyer Building, designed by Geo. B. Post, architect, situated at Cortland, Dey, and Church streets, New York,

FIG. 53.—HAVEMEYER BUILDING, N. Y.

towers fifteen stories, 193 feet including roof-house, above the level of the street; and is one of the most imposing and attractive of these majestic modern structures.

A building 60 by 215 feet and with its sides to the free outer air of the public streets gave room for the exercise of architectural imagination and invention, such as is seldom found in the business section of the city, where the transformation from low to high buildings is in progress.

All that the modern art of architecture and the modern skill in building craft could do to produce a building perfect in all its appointments has been employed to produce the Havemeyer Building.

Lime-stone, terra-cotta, brick and stone work, with wrought-iron riveted columns and steel beams, were the materials chiefly employed in its construction, and it is thoroughly fire-proof throughout. The floors, hall-walls, and stairs are of tile and marble, and fire-proof materials were used wherever possible.

Seven improved elevators are situated in the rear at the centre of the building, two of which are used as express elevators.

The roof floor is fitted as a restaurant and café, with open roof in warm weather and promenade, where superb views of the bay and surrounding country can be had from this height.

Floor Plan.—By referring to the floor plan of the building, Fig. 55, the arrangement of the floor area into offices, corridors, stairway, and elevators is shown. The longest front faces Church Street, the smallest Cortlandt Street, and the opposite side Dey Street. The rear is also open to the air.

An important feature has been introduced above the floor, in the way of extra light, secured by large windows in the street side and rear and glass windows in the partition enclosing the hall, making every room very bright and attractive.

The ground or main floor of the building has entrances running through from each street at the end of the corridor, with an additional stairway at each end connecting the basement and first floor. There is also a cellar in

addition to the basement below the street level. Each floor plan is divided into 22 offices, about 15'.6" × 19'.6" in size.

Wrought-iron Boiler Flue.—The boiler flue, 40 inches in diameter, is situated outside the building at the left side of the elevators, encased with brick the entire height of building. The brick shaft is 8'.6" outside diameter; a 24-inch wall is carried up 100 feet, a 20-inch wall 36 feet; then a 16-inch wall the remaining height, making a total of 197 feet from curb level to top of chimney. The circular wall of elevators is of brick and built up to within 4 feet of this total height. (See specifications and Fig. 61.)

Beam Plan.—We have in this example a variation of the skeleton construction; the walls simply carry their own weight and are not supported by girders and columns, as in the skeleton frame. The columns start in the walls at the lower story; when they reach the top of the building several feet intervene, as shown in the transverse section, at columns *D* and *K*, Fig. 55. The thickness of the rear brick wall in cellar 56 in., basement 48 in., ground story 44 in., 1st story 40 in., 2d story 36 in., 3d

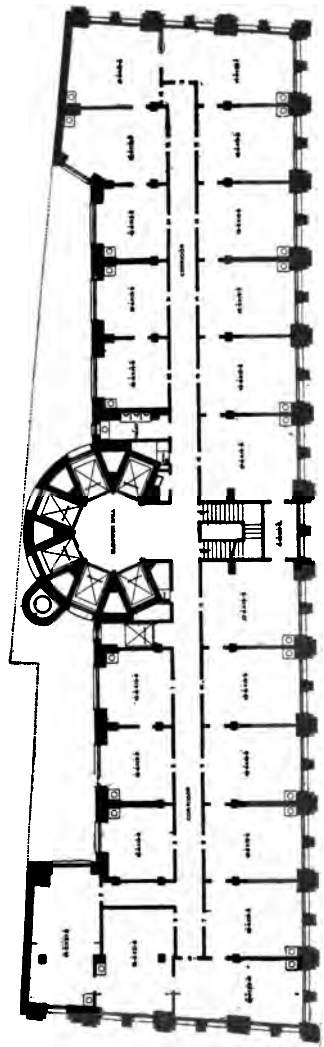


FIG. 54.—TYPICAL FLOOR PLAN, HAVEMEYER BUILDING

story 32 in., 4th story 32 in., 5th story 28 in., 6th to and including 9th story 24 in., 10th and 11th story 20 in., 12th story 16 in. The thickness of the front walls are: cellar 60 in., basement 56 in., ground floor 52 in., 1st story 44 in., 2d and 3d story 40 in., 4th story 39 to 36 in., 5th to and including 10th story 36 in., 11th story 32in., 12th story 30 and 28 in., 13th story and above 28 in.

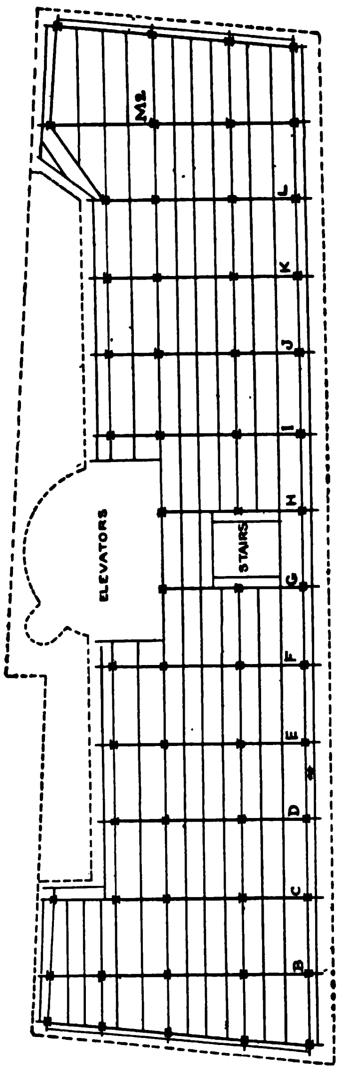


Fig. 55.—Beam Plan.

By comparing the floor plan, Fig. 54, with the beam plan, Fig. 55, it will be seen that the columns are placed in each partition; between the columns these partitions in most cases rest directly upon the girders. In calculating the floor loads the beams are not required to take this partition load, but it frequently happens that partitions are changed to suit tenants; it is therefore necessary that the beams be calculated to carry the total dead and live load.

The beams and girders being of steel are designed to support 200 lbs. per square foot of area, which includes dead and live load; the dead load equals about 100 lbs. per square foot. The girders on the *B* line of column are 12″ × 32 lbs. and 12″ × 40 lbs. per foot I-beams the same principle being used

throughout except at columns *D* and *K*, where lateral bracing is used (see the transverse section, Fig. 58, and details,

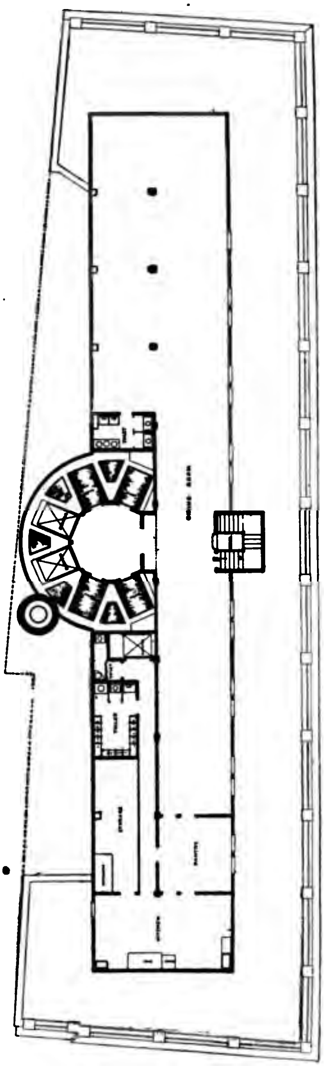


FIG. 56.—ROOF PLAN.

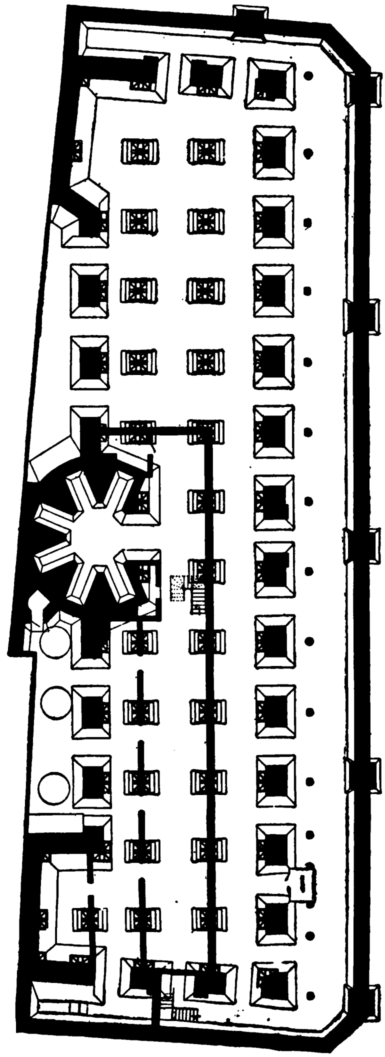


FIG. 57.—FOUNDATION PLAN.

Fig. 59); here two 12-in. channels 20 lbs. per foot are placed side by side.

The floor beams are 9 inches deep and from 21 to 27 lbs. per foot, placed from 3 ft. 10 in. to 4 ft. 5 in. centre and fastened to the 12-in. beam girders (see detail of column, Fig. 60, where the beam connection is shown). The columns are separated 16 ft. 1¼ in. from *B* to *C*, *C* to *D*, etc. From the street line (the dotted line shown in Fig. 55) to the centre of columns *B*, *C*, *D*, *E*, *F*, *G*, etc., is 4 ft. 4 in.; from *D* column to the column immediately in the rear on the *D* column line is 13 ft. 4 in.; to the next column 15 ft. 5 in.; to the next column on *D* line at rear wall is 9 ft. 5 in.; then to the area wall line 4 ft. 0 in. There are heavy anchors secured to the column at the lower floor levels, and built into the masonry as the work progresses, thus completely tying the column walls together; where the walls separate a few feet at the upper floor levels, short pieces of 12′ × 32 lbs. per foot I-beams were used, not only for anchorage, but to support the channels against the masonry, which in turn support the wall arches.

Each bay of floor beams were connected by one row of 1-in. diameter tie-rods, extending from wall facing street to rear walls, completely tying the masonry and metal frame together.

The transverse section also shows the depth of the foundations and their breadth.

The brick-work is about 24 ft. 6 in. below the ground floor level, and 9 ft. thick at the lowest point which rests upon heavy bed stone, then 6″ × 18″ planking, 2 layers thick, then concrete and piling. The foundations for the columns are built in like manner.

The foundation plan, Fig. 57, gives the position and arrangement of these columns and wall piers, also showing the portion of the rear wall which has a continuous foundation. The heavy continuous wall on the outside of the plate is the retaining wall of the street; the small piers between this wall and the piers of the long front are foundations for star columns made of four angles, back to back, supporting the basement floor under sidewalk, and a small wall from basement level to sidewalk.

To overcome the tendency to vibrate in the metal frame work of the structure when the work is in advance of the masonry and to keep the columns plumb, *sway braces* were introduced at the *D* and *K* line of column, as shown on the transverse section, Fig. 58, (*not lateral wind braces, as some have supposed*), made of $1\frac{1}{4}$-in. diameter rods carried from the basement to the twelfth story; where they were made of 1-in. diameter, and $\frac{3}{4}$-in. diameter in the thirteenth story.

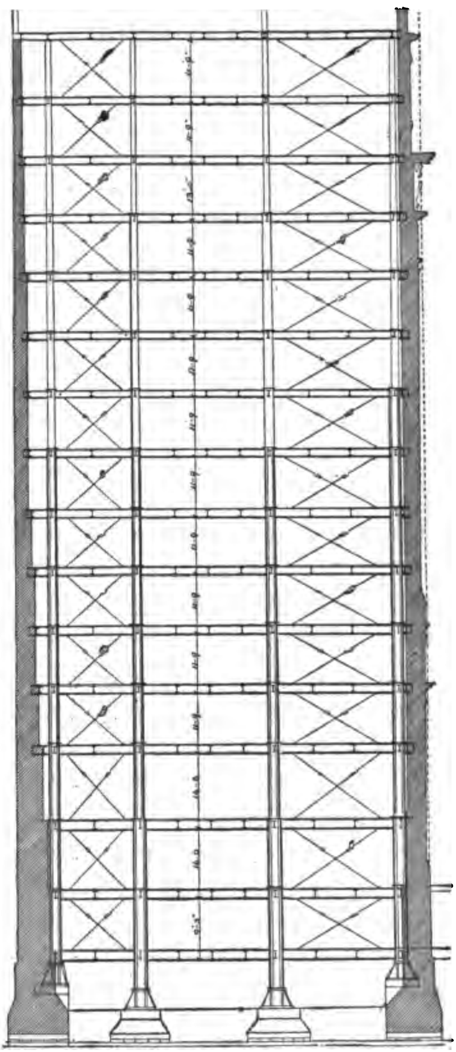


FIG. 58.—TRANSVERSE SECTION AT COLUMNS *D* AND *K*.

Each rod was made in two sections and each section had the ends upset for the eyes and the screw ends.

Turn-buckles were also provided for all the rods. The upset screw ends of the rods were $1\frac{5}{8}$-in. diameter for the $1\frac{1}{4}$-in. rods, $1\frac{3}{8}$-in. diameter for the 1-in. rods, and 1-in. diameter for the $\frac{3}{4}$-in. rods.

The sway-bracing was put in place and tightened as the work progressed. The detail, Fig. 59, shows the manner of connecting the bracing to the channel girder and columns.

To bind the masonry, three lines of continuous tie-bars were placed in the centre of the exterior walls, of 4″ × ¾″ flat wrought-iron bars, with welded ends and bored for 1½-in. diameter rods 16 in. long acting as a spear; the tie-bars were laid flat, while the spear was placed vertical in the masonry.

These tie-bars were placed at the 3d story, 11th story, and 13th story floor levels.

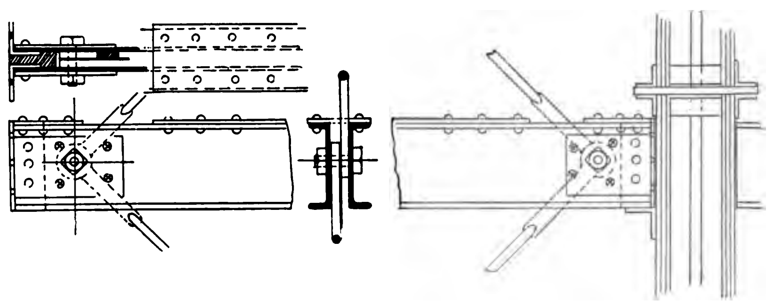


FIG. 59.

Column Detail.—The detail, Fig. 60, shows the manner of connecting the columns to each other and to the girders and floor beams of the building. In connecting the girders to the column one 6″ × 4″ × ½″ angle, 9 in. long was used and one 5″ × 4″ × ½″ angle 6 in. long for the 9-in. beams. The girder and beam were supported in addition to the above angle-knees by 4″ × 4″ × ½″ angle seats as shown.

The connection plates between the columns are of wrought-iron 1 in. thick, and when the upper columns are one or two inches less than the column below, two plates are used. To secure the upper with the lower column angle-knees are placed on each side and riveted through angles and plates.

The rivets in the body of the column as well as at the joints are 1 in. in diameter for columns with four cover plates (two on each side) and over, the thickness of plates being over ¾ in. thick; ⅞ in. diameter rivet for plate ¾ in. thick, and ¾-in. diameter rivets for plates less than ¾ in. thick.

In connecting the columns with the *cast-iron base plates*, the same general arrangement is observed as in connecting the columns to each other, as shown in Fig. 60; in place of rivets, bolts are used.

The base plates are $4' \times 4'$ and 2 ft. 6 in. height, the ribs

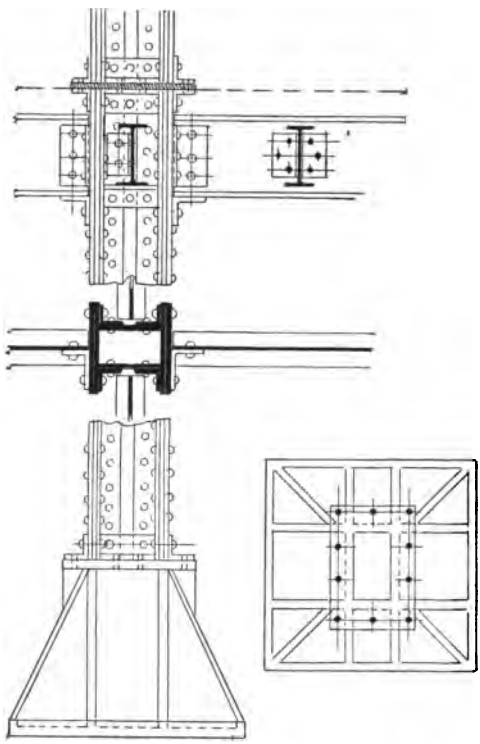


FIG. 60.

are as shown on the figure; the heavier columns have $1\frac{1}{2}$ in. and the lighter columns have $1\frac{1}{4}$-in. ribs.

The size and number of plates in a few of the lighter and heavier columns are given in the following table, with their loads :

BOX COLUMNS—HAVEMEYER BUILDING.

COLUMNS MARKED C, D, E, F, G, H, I, J, K, L.

	Length.		Outside Plates.		Webs.		Angles.		Load, Tons.
	Ft.	In.	No.	Size in In.	No.	Size in In.	No.	Size in In.	
Basement.....	9	3	2	15×¾	2	13½×¾	4	4×4×½	150
Ground floor..	16	0	"	"	"	"	"	"	139
1st story......	14	3	"	14×¾	"	9½×¾	"	3½×3½×½	128.75
2d "	11	9	"	13×⅝	"	9½×⅝	"	3×3×½	118
3d "	"	"	"	"	"	"	"	"	108.26
4th "	"	"	"	13×½	"	9½×½	"	"	98
5th "	"	"	"	"	"	"	"	"	88
6th "	"	"	"	13×⅜	"	9½×⅜	"	"	78
7th "	"	"	"	"	"	"	"	"	68.18
8th "	"	"	"	13×¼	"	9½×¼	"	"	58.39
9th "	"	"	"	"	"	6½×¼	"	"	48.6
10th "	"	"	"	"	"	"	"	"	38.8
11th "	13	0	"	"	"	"	"	"	28.9
12th "	11	9	"	"	"	"	"	"	19.14
13th "	"	"	"	"	"	"	"	"	9.36

1-in. diameter rivets were used to the 1st story, ⅞-in. to 2d story, ¾-in. to roof.

COLUMNS MARKED M2.

Cellar.........	9	0	4	15×1	2 "	11×1 11×15/16	4	4×5×13/16	345
Basement.....	9	3	4	"	" "	11×1 11×15/16	"	"	
Ground floor..	16	0	2 2	15×¾ 15×⅝	" "	11¾×11/16 11¾×⅝	"	"	321
1st story......	14	3	4	15×¾	" "	11⅝×⅝ 11⅝×11/16	"	4×5×¾	297.6
2d "	11	9	4	"	4	10×¾	"	4×4×½	274.75
3d "	"	"	4	14×¾	"	9½×¾	"	3½×3½×½	252
4th "	"	"	2 2	" 14×⅝	2 "	" 9½×⅝	"	"	229.4
5th "	"	"	4	"	4	"	"	"	206.9
6th "	"	"	2	14×1	"	9½×½	"	"	184.69
7th "	"	"	"	14×⅞	2	9½×⅞	"	"	162.46
8th "	"	"	"	14×¾	"	9½×¾	"	"	140.42
9th "	"	"	"	13×¾	"	"	"	3×3×½	118.45
10th "	"	"	"	13×½	"	9½×½	"	"	96.71
11th "	13	0	"	13×⅜	"	9½×⅜	"	"	75
12th "	11	9	"	13×¼	"	9½×¼	"	"	53.5
13th "	11	9	"	"	"	6½×¼	"	"	32.08

1-in. diameter rivets were used to the 4th story, ⅞-in. to 9th story, ¾-in. to roof

The following ideal specification covers fully all the requirements of the building:

Specification of the Wrought, Cast-Iron, and Steel Work, etc., for the Havemeyer Building.

Conditions.—The drawings and the specifications are intended to co-operate; but any work shown on the drawings, and not particularly described in the specifications, and any work evidently necessary to the complete finish of the building, as specified or shown, is to be done by the contractor without extra charge, the same as if it were both specified and shown.

The contractor is to comply with the corporation ordinances, the State, and other laws, and is to be liable for all penalties and all damages to life and limb that may occur owing to his negligence or that of his employés during the erection of the building.

No extra work will be allowed unless ordered by the architect in writing. No bill for extra work so ordered will be approved by the architect, unless it is rendered immediately upon completion of the said work.

Sub-contracts are to be submitted to the architect for his approval, and in no case will any work be accepted furnished by sub-contractors not approved of.

The architect will furnish drawings exhibiting the work to be done by the contractor, and will also furnish detailed drawings for all moulded, carved, and ornamental work; and any work made without or not in strict conformity with such drawings, or differing from the requirements of the drawings and the specifications, will be rejected, and must be removed and replaced by work in conformity with the requirements of the drawings and the specification, and all work injured or destroyed thereby must be made good at the contractor's expense.

Shop drawings, copies of the architect's drawings furnished, templets, reverse templets, patterns, models, and all

necessary measurements at the building are to be made by the contractor at his own expense.

Figured dimensions on the drawings are to be followed in all cases in preference to scale-measures.

The contractor must procure all necessary permits in connection with his work and pay all fees for the same.

The owner, through the architect, reserves the right to annul and cancel the contract in case the contractor neglects or refuses to remove rejected work and to replace the same as above specified, and according to the instructions of the architect, within a reasonable time after having been notified.

A schedule in detail of the prices on which the contract is based is to be furnished to the architect on signing the contract, which schedule will be the basis for all payments on account of the contract.

The contractor must properly protect his work from injury until the final completion of the building and the acceptance of the same.

Any damage done to work of other contractors by the contractor, his sub-contractors, and employés will be made good at the contractor's expense.

Time of Completion.—All the work is to be completely finished on or before the 1st day of July, 1892; the entire second tier of beams must be set on or before the 1st day of August, 1891; the roof tier of beams must be set on or before the 4th day of January, 1892, and all the staircases and elevator fronts must be finished on or before the 1st day of March, 1892.

The work included in this specification must be prepared and erected in its various stages at such times as may be necessary to complete the same and parts of the same at the times mentioned, and without interfering with or delaying the progress of the work of other contractors.

The contractor is to provide all necessary night and overtime work without extra charge.

Should any delay occur in the progress of the work included in this specification, or should the work of other contractors be delayed on account of delay in the iron-work or on account of replacing or altering defective or rejected work, the contractor is to pay to Theo. A. Havemeyer, Esq., the sum of two hundred and fifty dollars ($250.00) as liquidated damages for each and every day that the work is delayed, and for each and every day that the iron-work may be unfinished and uncompleted after the 1st day of July, 1892.

Payments will be made only on the certificate of the architect.

On or about the first day of each month a certificate will be given by the architect for a payment on account of the contract of eigty-five per cent (85%) of the value of the work furnished and put up at the building, provided the contractor has made application over his signature on blanks furnished by the architect on or before the 25th day of the preceding month, and that a schedule has been furnished, as before specified.

A certificate for the balance will be given by the architect upon the completion of the contract, in conformity with the drawings and the specification, application having been made as before specified.

Certificates will be given by the architect, at his option, on or about the 1st day of each month for a payment on account of the contract of sixty-five per cent (65%) of the value of finished work, set aside and stored as hereinafter provided, applications for the same having been made as before specified.

No certificate will be given in case any work is furnished not in strict conformity with the drawings and specifications, and until defective work has been removed and replaced as specified, and to the satisfaction of the architect.

Any certificate given or any payment made on account of the contract for work furnished and erected in the building, or for materials finished and set aside, does not act as an acceptance of any materials or work which may subsequently be

found to be defective by reason of existing defects at the time such certificate is given or payment made, or defects arising from accidental injury or otherwise, until the completion of the contract.

The contractor must replace all such defective work on which payments have been made before another certificate will be issued.

Sub-contract.—The contract for the iron-work will be made at the option of the owner, a sub-contract to the general contract for the erection of the building, and all payments in the manner described will be made by the general contractor, and the contractor will be liable to the general contractor for the proper performance of the contract, and will not be relieved of any of the obligations herein specified.

Materials and Workmanship.—All materials, of every kind and description, are to be of the very best quality; and all work necessary to the complete finish of the iron-work, as shown on the drawings and as directed by this specification, is to be executed in the most thorough, substantial, neat, and workmanlike manner, to the entire satisfaction of the owner and the architect, to whom every facility is to be given by the contractor for inspecting the work as it progresses. All tests, as required by the law, are to be made, the contractor paying all expenses.

The contractor is to furnish all necessary materials and labor, and is to provide all tools, derricks, scaffolding, planks, runs, horses, and all necessary mechanical appliances for properly prosecuting the work. All necessary freights, cartages, and transportation and all handling of materials must be paid by the contractor.

Delivery and Storage.—The contractor is to commence delivery and erect his work as soon as the building is ready to receive the same, and must continue delivering and erecting as rapidly as possible, without interfering with or delaying the work of other contractors.

After the second tier of beams has been set the iron-work of the various floors must be carried up in advance of the mason work, so that one tier of beams and two tiers of pillars or columns are set ahead of the mason-work, story to story.

The site and the building are not to be used by the contractor for the storage of materials.

All finished work set aside ready for delivery, on which a payment on account of the contract is desired, is to be stored and set aside on premises properly housed and protected at the expense of the contractor.

A lease of these premises is to be executed by the contractor to Theo. A. Havemeyer, Esq., for the time the finished work remains stored therein. The contractor is to be liable for all injury to the finished work when so stored.

The contractor at his expense must insure against loss or damage by fire all finished work so stored, and assign the policies of insurance to Theo. A. Havemeyer, Esq., as security for the advance that may have been made thereon, and also for the performance by the contractor of his agreement to repair such damage.

The contractor is to keep the policies in force until the finished work so stored shall be delivered at the building.

Wrought-iron.—All wrought-iron must be tough, fibrous, and of uniform quality, straight and smooth, free from cinder pockets or injurious flaws, buckles, blisters, or cracks. The tensile strength is to be 48,000 to 50,000 pounds per square inch of section, with an elastic limit of 24,000 to 26,000 pounds per square inch of section; the elongation not less than 12 per cent. The higher limits will be required for the rivets, angle plates, and knees.

All wrought-iron must, when cold, bend without cracking through 180 degrees to a curve whose diameter is not over twice the thickness of the specimen. When nicked and bent its fracture must be nearly fibrous, showing but few crystalline spots.

Steel.—All steel is to be mild, and must be of uniform quality, straight and smooth, free from flaws, cracks, or other defects. The tensile strength is to be 60,000 pounds per square inch of section, with an elastic limit of 28,000 to 30,000 pounds per square inch of section; the elongation not less than 20 per cent.

All steel must be capable when cold of bending double, flat, without sign of fracture on the convex surface of the bend.

Cast-Iron.—All castings must be of the very best quality, tough, gray iron, free from defects. No scrap-iron is to be used or mixed with cast-iron.

All the castings are to be true, smooth, and straight, of a uniform thickness of metal, and must be entirely free from blow-holes, honey-combs, cinders, seam-marks, and other defects.

Tests.—All materials used in all stages of manufacture shall be subject to full inspection and test by the architect, and the contractor is to supply all requisite facilities for such inspection and tests without charge. The architect shall have full power to reject any of the materials or parts, if in his judgment such materials are not in conformity with the specifications, at any time before the final completion and acceptance of the work; and his decision shall be final. All samples and specimens are to be prepared by the contractor, and all tests are to be made at his expense. Such tests will be under the direction of the architect, who shall be the sole judge of how many are necessary, and in what manner they shall be made.

Construction of Work.—Where work is specified as "good and sufficient," or where the work is not fully explained by the drawings, the contractor shall in all cases, before the execution of the work, submit to the architect for his approval a detailed specification for the same. The architect shall be at liberty to alter and amend such specification, if in his opinion the work as described is not of materials, proportions, and workmanship best adapted to the purpose.

The minimum weights and sizes of all girders, beams, pillars, columns, rods, bolts, rivets, etc., are shown on the drawings and given in this specification; and any materials delivered at the shops or at the building which are not of the prescribed weights and sizes will be rejected by the architect, to whom every facility is to be given by the contractor for the purpose of examining the work, both at the shops and at the building; the contractor is to do all necessary handling and weighing without extra charge.

The screw threads of all rods and bolts are to be carefully cut, so that all portions of the same will be formed of solid iron. No bolts or rods are to be pieced or welded together.

The ends of the bars and rods used for bracing are to have the screw ends for the turn-buckles upset, and all threads are to be carefully cut.

Nuts are in all cases to be extra heavy. Sufficient washers of proper sizes and shapes are to be provided. All rods used for braces, counters, and sway-rods are to have turn-buckles. All eye-bars are to have the ends upset, and are to have the pin and bolt holes bored. All pin and bolt holes are to be accurately bored at right angles to the axis of the members, and must not be more than one-thirty-second of an inch larger than the diameter of the pin or bolt. All holes in cast-iron are to be bored.

All riveted work is to be done in the very best manner. Rivet-holes may be drilled or punched, and must not be more than one-sixteenth of an inch larger than the diameter of the rivet; if punched, the work must be carefully done with well-proportioned punches and dies, so that the holes shall be straight, clean, and sharp, and that the rivets can be driven without *drifting, which will not be allowed.* When the pieces to be joined together are assembled, the holes through the parts for each rivet shall be strictly in line and normal to the surfaces; if a slight misfit should occur the holes must be truly reamed to the required size and larger rivets must be used.

After the riveting is done, the pieces joined together shall be closely in contact throughout, and the rivets shall be tight; fill the holes perfectly and have full semi-spherical heads concentric to the rivet-holes. Rivet-heads are to be countersunk wherever necessary for bearings. All rivets are to be made of the very best refined iron. The pitch of rivets shall not exceed 6 inches, nor be less than three diameters of the rivet. Rivets $\frac{3}{4}$-in., $\frac{7}{8}$-in., and 1-in. diameter will be used. Except for bars less than $2\frac{1}{2}$ in. wide, the distance between the edges of any piece and the centre of the rivet-hole must not be less than $1\frac{1}{4}$ in.; where practicable it shall be at least twice the diameter of the rivet. All joints in riveted work must be fully spliced, and the ends before splicing must be dressed straight and true, so that there will be no open joints.

All the beam framing throughout is to be riveted, and the fastenings to the pillars and columns are to be riveted. All the pillars and columns are to be riveted, pitch to be 4 in., and in all cases the five end rivets are to have the minimum pitch. All brackets and lugs on the pillars and columns are to be riveted.

Box girders are to be riveted in the same manner; the plates for the flanges and webs are to be riveted together by means of angle-irons; pitch of rivets is to be 4 in. Heavy cast-iron filling pieces, with lugs and flanges, fitting accurately the flanges and webs, are to be placed at the ends and 4 feet apart, through which the webs are to be bolted together with $\frac{3}{4}$-in. bolts, not over 6 in. between centres.

For the bearings of the beams, angle-irons, riveted to the webs are to be provided.

All riveted work must be perfectly true and straight.

All knees, angles, and connecting parts in the various framings are to be of wrought-iron, all well riveted and bolted. No cast-iron is to be used. All angle plates and knees are to be bent while hot, and across the fibre of the metal; when finished

they shall be free from cracks and seams, without initial strain, and with the full strength of the plates preserved.

All the girders between the columns on all the stories (except on the second tier between *G* and *G*2, *H* and *H*2) are to be set so that the bottom flange is 1½ in. below the bottom of the beams and the beams framing into the same are to be cut off square to fit the web of the girders. In all cases wedge foot-blocks on which the beams rest are to be provided, riveted to the bottom flange of the beams or girders; the bearing in all cases must be uniform without wedge or blocking. Where beams are framed into headers and trimmers they are to be flush on the bottom, and the ends of the beams are to be cut to fit accurately the shape of the beam on which they rest. All beams throughout must be secured in position by angle-irons and rivets. The angle-irons are to be 3½ in. × 3½ in. and as long as the web of the beams, and placed each side of same. Six ¾-in. rivets are to be used for all beams 10 in. or less in depth, and nine ¾-in. rivets for all other beams. Beams and girders are to be secured in the same manner to the pillars, columns, and plate girders; channel bars are to be framed in the same manner.

Girders formed of two or more beams are to have inserted between the webs of the beams, not more than 6 feet apart, and at the ends blocks of cast-iron fitting accurately the form of the beams cast with proper lugs and flanges, through which the beams are to be bolted together by two ⅞-in. bolts for all beams over 10 in. in depth, and one bolt for all beams of 10 in. and less depth.

The girders of all the tiers of beams located between the columns *D* and *D*1, *D*2 and *D*3, *K* and *K*1, *K*2 and *K*3 are each to be made of two 12-in. channel bars, 20 lbs. per foot each. These girders are to have wrought-iron packing pieces placed between the webs opposite the ends of the beams framing into the same, and through which the rivets are to pass. The ends of these girders are to be fastened to the pillars or

columns by means of angle-irons placed back to back with packing pieces between them, riveted to the pillars or columns and girders. The web of each channel bar (of the girders) at the end where the pin-hole for the bracing is located is to be reinforced by a ½-in. thick wrought-iron plate 9 in. × 12 in., secured by six ¾-in. rivets.

All girders formed of channel bars are to have ⅜-in. thick and 8-in. wide top plates riveted to the flanges with ¾-in. rivets, pitch 5 in. These top plates are to be of sufficient length, extending within ½ in. of the cross-bracing.

All beams and girders carrying walls are to have ⅜-in. thick iron plates, the full width of the wall, except otherwise directed, securely riveted to the upper flanges; rivets to have 8-in. pitch and countersunk heads. Short plates are to be provided near the ends of the beams from *A* to *F* and *G* to *N* on the basement tier, and circular plates under the area wall opposite piers *A* and *N*.

The bearings on the walls of all beams, girders, and lintels must be full and true, not less than 6 in. at each end. Beams having a span of over 20 feet, and girders, are to have bearings on the walls of not less than 10 in.; lintels over openings more than 5 feet wide are to have 1 inch additional bearing for each additional foot of span.

The construction of the pillar and column connections, the base plates and the other work, is to be made as hereinafter specified.

The substitution of pipes, thimbles, or other devices for filling pieces, blockings, packing pieces, etc., as specified, will not be permitted under any circumstances.

The contractor is to do all drilling and tapping of iron-work that may be required, and is to furnish all necessary machine screws and bolts for fastening wood and other work.

Setting.—All the iron-work throughout is to be set in the very best manner; all bearings are to be full and true—the contractor providing all necessary scaffolding, trestling, centres,

shores, and braces that may be required. In all cases the work must be bolted and riveted together as it progresses. All beams, girders, and lintels are to be set level, and the beams and girders of the staircase opening, shafts, and well-holes are to be set plumb over one another. All columns are to be set perfectly plumb on true beds.

All iron-work, comprising fascias, railings, and other work connected with the masonry, must be well fitted, and the joints are to be leaded.

A sufficient number of men must be kept at the building, at the contractor's expense, to do all necessary cutting, fitting, and drilling that may be required.

Painting.—All the iron-work is to be properly and thoroughly cleaned from rust and cinders, and is to receive one coat of metallic paint and pure linseed oil, well brushed in. A good coat of the best light boiled linseed oil is to be applied to all surfaces in contact before they are riveted together. The paint is to be allowed to dry before the work is delivered at the building.

The cast-iron lintels for the windows in the rear walls are to be painted with two good coats of red lead, allowed to dry before they are set.

After the work has been set, it is to be painted another coat of the same kind of paint before it is built in. Prince's metallic paint or Rossie iron ore paint is to be used. Pure linseed oil only is to be used for the paint.

All cast-iron work exposed to view is to be well filled, and rubbed down to a smooth surface before the paint is applied.

Beams and Channel Bars of such sizes and weights as shown on the drawings, and as hereinafter specified, are to be provided and set for all the various tiers, and wherever necessary and required to form proper supports for the floor arching, vault arching, and roof arching. All beams, channels, etc., not shown are to be "good and sufficient."

All beams supporting the elevator sheaves will be provided under another contract.

The ends of all girders, beams and channel bars resting on pillars or columns are to be riveted at each bearing with two $\frac{3}{4}$-in. rivets through the flanges, and the girders of all tiers of beams are to have two $3\frac{1}{2}'' \times 3\frac{1}{2}''$ angle irons riveted to the pillars or columns, and to the webs of the girders with $9\frac{3}{4}$-in. rivets. The ends of girders formed of channel-bars are to be riveted at the flanges, and with angle irons with packing pieces as before specified.

The beams between *A*2 and *C*2 on the second tier are to have $3'' \times 3''$ angle irons, riveted to the web to receive the high level floor arching.

At the foot of the main stairs (ground floor) between *G*1 and *H*1 two beams are to be provided, placed over one another, for the high and low level floor arching.

The bent channel-bar for the floor arching of the elevator halls on the first story and on all stories above is to be carried at each pier on 6-in. beams, 4 ft. long, to which it is to be riveted with two $\frac{5}{8}$-in. rivets to each beam, and the portion over the entrances is to be hung by means of three $3\frac{1}{2}'' \times \frac{1}{2}''$ stirrup-irons, properly shaped and riveted.

Over the bent channel-bars at all the elevator door openings, to form a support for the saddles, provide and set 6-in. beams, 16 lbs. per foot, properly supported on upright beams of the same kind, placed on top of the cantilever beams carrying the bent channel-bars, all to be properly framed, fastened, and riveted.

The beams for the deck-house roof are to be of such sizes as hereinafter specified.

Channel-bars not less than 8-in. deep are to be provided in all cases for supporting the floor, vault and roof arches at the walls and piers, and are to be set with a 2-inch bearing on the walls the full length.

All beams and channel-bars are to have properly drilled or punched rivet-holes and holes for tie-rods.

All beams and channel-bars are to be of mild steel of the standard weights and sizes made by Carnegie, Phipps & Co., Ltd., Pittsburgh, Pa.; or of other equally good make, of the same or larger sectional areas for beams and bars of the same depth.

Beams and channel-bars of less sectional area in the flanges, or of lighter weights than those above mentioned, will not be accepted under any circumstances; and if furnished, set, and fastened, they must be removed promptly upon notification from the architect.

In case the contractor cannot furnish steel beams of the kind required, he shall be at liberty, with the approval of the architect, to furnish wrought-iron beams and bars, the moments of inertia of which are larger than those of the corresponding steel beams and bars, and calculated with an ultimate strain not exceeding 15,000 pounds to the square inch. All such beams and bars are to be "good and sufficient," and the contractor is to furnish to the architect for his approval a detailed list of all such beams and bars before work is commenced.

Girders made of two or more beams, as before specified, are to be provided as shown on the drawings.

Box Girders, made as before specified, are to be provided and set as shown on the girders.

The following box girders will be required for supporting the tank-house floor beams:

Location.	Depth.	Thickness of webs.	Size of Flanges.	Angle Irons.	Rivets.
G2–G4	15″	$\frac{1}{2}$″	13″ × $1\frac{5}{8}$″	$3\frac{1}{2}$″ × $3\frac{1}{2}$″ × $\frac{1}{2}$″	$\frac{7}{8}$″
H2–H4	15″	$\frac{1}{2}$″	13″ × $1\frac{5}{8}$″	$3\frac{1}{2}$″ × $3\frac{1}{2}$″ × $\frac{1}{2}$″	$\frac{7}{8}$″

Tie-rods.—Are to be not less than one inch in diameter, and are to be provided for all beams and channel-bars.

All are to be placed in continuous rows, at right angles with the webs of the beams and channels.

All beams are to have one row of tie-rods, placed equidistant between the bearings. The ends of all beams and channel-bars, not sufficiently fixed laterally by iron or mason-work, are to have additional tie-rods; and additional tie-rods are to be provided wherever required.

All tie-rods are to be properly tightened before the arches are built.

Anchors, Straps, Clamps, etc.—Provide and deliver at the building all necessary anchors, clamps, and dowels for the stone-work; each piece of stone is to be anchored, and all copings of wall and boiler-flue stack are to be clamped. All are to be made of $1\frac{1}{2}'' \times \frac{5}{8}''$ wrought iron, of proper length, and of such shapes as may be directed.

Provide for the mason all necessary anchors for the corner piers A, A4, N, and N3, which are to be anchored on every story (except where continuous tie-rods are hereinafter specified) as follows: pier A to A1 and B, pier A4 to A2, pier N to N1 and M, pier N3 to N2. These anchors are to extend from centre to centre of the piers, and are to be made flat of $2\frac{1}{2}'' \times \frac{1}{2}''$ iron with a $1\frac{1}{2}$ in., 16-in. long spear at each end.

Provide for the mason all necessary anchors for anchoring the corners of the walls, every four feet, to be made of $1\frac{1}{2}'' \times \frac{3}{4}''$ iron, 4 feet long, with a 1-in. diameter 16-inch long spear at each end. These anchors are to be used where the walls and piers are not carried up at the same time.

Provide for the mason 20 anchors for each story from ground floor to the 13th story, for anchoring the exterior circular wall to the interior piers, these anchors are each to be about 8 ft. long and made of $1\frac{1}{2}'' \times \frac{3}{4}''$ iron with a 16-in. long $1\frac{1}{4}$-in. diameter spear at each end; at the piers one spear may be used for two anchors. These anchors are to be set edgewise against the face of the thin walls on the machine shaft side.

Provide for the mason flat tie anchors for the walls enclos-

ing the elevator shafts at G2 and H2; these tie anchors are to be made of 4″ × ¼″ iron with 16 in. long, 1 in. diameter spears, and are to be placed two in the height of every story.

Provide for the mason all necessary anchors, rods, bolts, cantilevers, and clamps for the terra-cotta work. All the trimmings of all façades, above the stone-work on the ground floor, will be terra-cotta.

Each projecting piece, arch-blocks, caps, etc., will be anchored, and plates and rods are to be provided for the balusters. Cantilevers are to be provided for the four cornices, including the blocks, over the caryatides on the 11th story. The cantilevers for the main cornice are to be made of 4-inch beams, placed about 2 feet apart (one in each block); a 6-in. beam extending from pier to pier is to rest on top of the inner end of the cantilevers or is to be framed to the same; cantilevers of 4″ × 4″ tee-irons, placed about 2 feet apart, are to be provided for the three other cornices.

Anchors are to be made ¼″ × 1″ and ⅜″ × 1¼″ iron, of such shapes and lengths as may be directed. Rods for balusters are to be ⅝ in. diameter, with necessary plates, washers, and nuts —all as may be directed.

All anchors, clamps, dowels, and rods above specified are to be galvanized.

Provide for the mason all necessary lengths of ½″ × 2″ bar iron for man-hole openings in fire proof partitions and furrings, as may be directed.

The ends of all beams and girders resting on a wall or pier are to have 1¾″ × ½″ iron anchors, each bolted to the web with two ⅝ in. bolts, and the other end turned around a 1-in. diameter wrought-iron spear 16 in. long, set vertically, and in no case lessthan 4 inches from the end of the beam or girder.

Beams and girders resting on tops of pillars, colums, and girders, where they abut or are placed opposite one another, are to be strapped together by means of 1¾″ × ½″ wrought-iron

straps, of proper lengths, riveted to the end of each beam and girder with two ¾-in. rivets at each end.

The channel-bars are to be provided with bent anchors made of ¾″ × ½″ iron, bolted to the ends of the tie-rods.

All necessary anchors are to be provided for the light cast-iron work hereinafter specified.

Tie-bars.—Provide and set three lines of continuous tie-bars, placed in the centre of the exterior walls, anchored with a 1½-in. diameter 16-in. long spear at each pier, extending entirely around the building. One set of tie-bars is to be located on the line with the 3d story floor, one set on the line with the 11th story floor, and the other set is to be on the line of the 13th story floor on all the rear walls and over the 13th story window arches on all the front walls. These tie-bars are to be made of 4″ × ¾″ iron, with upset ends and the holes are to be drilled.

Sway-braces.—Diagonal sway-braces are to be provided and set between the ends of the girders, located between D and D1, D2 and D3, K and K1, K2 and K3, on each story from the basement girders to the roof girders.

All the rods used for the sway-braces are to be 1¼-in. diameter except those on the 12th story, which are to be 1-in. diameter, and those on the 13th story, which are to be ¾-in. diameter. Each rod is to be made in two sections, and each section is to have the ends upset for the eyes and the screw ends.

Turn-buckles are to be provided for all rods. The upset screw ends of the rods are to be 1⅝-in. diameter for the 1¼-in. diameter rods, 1⅜-in. diameter for the 1-in. diameter rods, and 1-in. diameter for the ¾-in. diameter rods; in all cases the screw threads are to be carefully cut. The eyes of the rods are to be properly made with upset ends, having full sectional areas; the pin holes are to be bored; the 1¼-in. and 1-in. diameter rods are to have the ends flattened to ⅞″ thickness.

All the pins are to be 1½″ diameter except those of the 13th

story girders, which are to be $1\frac{1}{4}$-in. diameter, and those of the roof girders, which are to be 1 in. diameter.

All pins are to have proper heads, nuts, washers, and packing pieces.

All sway-bracing must be put in place and properly tightened as the erection of the work advances, story by story.

Lintels of Cast Iron are to be provided for all openings in the masonry, except otherwise specified or directed. The flanges are to be the full width of the walls and piers, except for openings in the walls of the façades on the three streets, which are to be the width of the backing. All window openings with square and round heads are to have cast-iron lintels, and in all cases the lintels for double windows are to be made to extend over both windows. The lintels for openings in curved walls are to have the outside flanges made on the curve and the webs straight.

The lintels for the windows in the rear walls are to be made with a rebate for receiving the heads of the window frames. The exposed part of these lintels are to be finished smooth and clean.

In all cases where the window openings have flat arches, the lintels are to be placed over the same as shown and as may be directed.

The inner lintels of windows in elevator shafts and of the openings to the pipe shafts on the 2d and 4th stories are to be drilled and tapped, and large brass screws are to be provided for fastening the marble soffits.

All lintels exposed to view are to have the exposed parts finished.

The webs of all lintels are to be four inches high at the ends, and are to have a rise of 1^1 inches for every foot of span.

Lintels are to have skewbacks wherever required.

All lintels over 16 inches wide are to have two webs, or lintels are to be placed side by side.

All lintels (not carrying floor beams) over openings not

over 6 ft. wide are to have 1-in. thick flanges and webs, and over 6 ft. and not over 8 ft. they are to be 1¼ in. thick, and over 8 ft. 1½ in. thick. In special cases the lintels are to be "good and sufficient."

Pillars of Wrought Iron.—All the pillars throughout for supporting the girders, beams, and other parts of the building are to be made of 4 wrought-iron plates, riveted together by means of 4 angle irons. All the plates and angle irons are to be perfect in every respect; all finished with clean, sharp edges.

In all cases the girders and beams are to rest on 4 in. × 4 in. × 1½ in. angle iron brackets, riveted to the pillars with not less than four ¾-in. rivets for those under the girders and three ¾-in. rivets for those under the beams. The ends of all beams and girders are to be riveted to these brackets with two ¾-in. rivets. In addition, the ends of the webs of all beams and girders are to be riveted to the pillars with angle-irons as before, specified.

All pillars are to have cast-iron base plates, hereinafter specified.

The ends of all pillars are to have four 3 in. × 3 in. × ¾ in. angle brackets, riveted with not less than two ¾-in. rivets.

The ends of all pillars, after the angle brackets have been riveted on, are to be turned off in a lathe at right angles with the axis of the pillars.

Wrought-iron plates, faced off on both sides, not less than one inch thick, finished, are to be placed between the ends of all pillars; the faces are to be parallel to one another, and the plates are to be of sufficient size to afford full bearings for the angle brackets; the plates of the pillars next to the masonry piers are to have a sufficient projection beyond the angle brackets for securing the wall anchors. In all cases where a pillar of smaller dimensions is placed on a larger one, an additional plate of the same kind and thickness is to be provided.

The tops of the pillars are to have angle brackets and ½-in. thick plates.

The bearings of all pillars throughout are to be full and true; no shimming or wedging will be allowed.

In all cases where necessary, packing pieces are to be used back of the angle brackets.

All pillars are to be riveted together through the angle brackets and plates, with not less than six ¾-in. rivets.

All pillars at the exterior piers are to be securely anchored to the mason-work with special anchors, made of 5 in. × ⅝ in. iron, riveted with two ¾-in. rivets to the under side of the plates, extending to the centre of the piers and turned around at 1½-in. diam., 16-in. long spar.

These anchors are to be galvanized.

The pillars B1, B2, B3, C1, C2, D1, D2, E1, E2, F1, F2, G1, H1, I1, I2, J1, J2, K1, K2, L1, L2, M1 and M2 are each to be made in one length, extending through the cellar and basement stories.

16. Pillars in vaults composed of 4—3½ in. × 3½ in. × ½ in. angle-irons ¾-in. rivets, riveted together in shape of a +.

Posts.—The two posts carrying the tank-house floor at G3 and H3 are to be made of 12-in. steel beams, 40 lbs. per foot; the girders and beams resting on the same are to have angle brackets and knees, all perfectly riveted.

The posts in the east-vault walls in cellar and basement shown on the plans are to be made of 10-in., 33 lbs. per foot, beams, each in one piece, and all are to have base-plates and 10-inch channel-bars framed to the top. All connections are to be fitted, framed, and riveted the same as specified for beams.

All other posts that may be required for supports are to be made of beams "good and sufficient."

Cast-iron Base-plates are to be provided for all the pillars and columns in the cellar, as shown. These base-plates are to be cast with flanges, ribs, webs, and rings, as shown; the thickness of the shell and of the bottom and top flanges is to be 1½ in., and the other parts not less than 1¼ in. thick.

The castings are to be perfectly sound. The bottom flanges and rings on which the columns rest are to be faced off in a lathe parallel to each other. All base-plates are to be bedded solid in Portland cement on the granite caps. The base-plates of the pillars and columns G4, H4, are to be made as shown; the metal is to be 1 in. thick.

The base-plates under the posts and under the pillars supporting the area wall are to be 1½ in. thick.

Roofs.—The main roof is to be built as shown on the deck-house plan. The roof of the deck-house is to be built as follows: the pillars on columns C2, D2, F2, G2, H2, I2, J2, K2, and L2 are carried up supporting a girder 10 in. deep, 33 lbs. per foot. Soffits are to be prepared with necessary angle-irons, prepared to receive the necessary wire lathing.

Staircases.—All the staircases throughout are to be built of iron, with iron, marble, and slate steps and platforms, as hereinafter specified. Strings are to be made of wrought and cast iron, with wrought and cast iron ornamental face-work; wall-strings are to be of cast-iron with ornamental face-work, accurately fitted and properly supported. In all cases the ends of the wall-strings are to be properly made to fit the base, and the under side is to finish properly at the plastering.

In all cases the floor beams at the landings are to be covered with faciæ and moulding extending from the walls to the newel-posts and between the newels. All the work to be ornamented and finished on both sides. The under side of the platforms and landings are to be finished with ornamental ribs and panels. All the platforms of the main staircase and the platforms of the other staircases are to be supported on every story by means of four 3-in. angle or tee-irons placed vertically and extending from the tops of the beams to the bottoms of the beams above, to which each is to be bolted at each end with the necessary angles and knees and six ⅝-in bolts.

Newels are to have caps, drops, panelled, moulded, and ornamented shafts, as shown and as may be directed.

In all cases the staircase walls are to be made to correspond with the staircase strings, the full thickness of the floor.

All risers are to be of cast-iron, moulded on both sides. The starts of all staircases, where directed, are to have moulded and cast-iron jib panels, vertical panels, and soffits, as shown and as directed.

The steps from the low to high level on the ground floor in the Church Street entrance hall are to be built with concealed strings at the ends and two intermediate strings.

All the railings are to be made of wrought and cast iron. The posts and balusters are to be of polished wrought-iron; those of the main staircase are to be twisted. All are to be provided with ornamental cast-iron sockets and panels, and the top-rail is to be of wrought-iron, prepared to receive the wood hand-rail, except the railings of the stairs to the cellar near A2 and G1, which are to be made as hereinafter specified.

The horizontal railings of the staircases are to be made in the same manner, to correspond.

The steps from the low to the high level on the ground floor, the main stairs from the ground floor to the thirteenth story, the stairs from the thirteenth story to the deck-house, and the two staircases from the basement to the first story are to have treads, platforms, and landings of the very best quality of pink Knoxville marble 1½ in. thick, finished with nosings on the fronts and with nosings on the returns, except the enclosed steps, which are to have nosings on the fronts only. Each tread is to be in one piece, and the platforms are to be made in one, two, and three pieces each. In no case are the pieces to be less in width than the stairs by the width of the platform. The platform of the landing at the deck-house floor is to be made in one piece.

All steps and platforms are to be securely fastened by means of screws and leaded holes.

All the marble-work must be delivered in perfect condition, free from scratches, spalls, and other defects.

The risers of the two steps in the Church Street entrance hall are to be of the very best quality polished Alps green marble, set and fastened in the very best manner.

All the marble steps and platforms are to be polished in the very best manner on all sides except the tops, which are to be rubbed.

All joints are to be made uniform and close, and are to be properly pointed.

On completion of the building all the marble-work is to be oiled one good coat of pure linseed oil, well rubbed down.

The two staircases from the cellar to the basement story near A2 and G1 are to have the treads and platforms made of cast-iron $\frac{1}{2}$ in. thick, diamond pattern, with nosings, and the risers are to be cast with segment openings, all properly fitted and bolted to the strings. Proper supports and hangers are to be provided for these staircases. The railings are to be made of $\frac{3}{4}$-in round iron, bolted to the strings, properly braced, to have a wrought-iron top rail and moulded capping. The newels are to be cast-iron, plain moulded, with caps and drops.

As soon as the iron-work of the staircases has been set, the same is to be covered with boxing, and temporary wood treads and platforms are to be provided, all properly fastened. After the marble and slate treads and platforms have been set the temporary wood treads and platforms are to be replaced, to protect the marble and slate-work, and must be kept in repair until the completion of the building.

Ladders made of wrought-iron, strings 3 in. $\times$ $\frac{1}{2}$ in. with double rungs of $\frac{3}{4}$-in. round iron, are to be provided and put up as follows:

In each of the six elevator-machine shafts at the second and fourth stories, in the deck-house to the tank-house floor, and one to the grating over the service elevator.

All the above ladders are to be securely fastened, and are

to be put up complete with necessary 1-in. diameter hand-rails, as may be directed.

Railings.—All railings are to be made of wrought-iron, to be securely fastened and braced. The ornamental scroll-work is to be well made and firmly riveted.

On the streets railings are to be provided for the store entrance steps, as shown and as may be directed. The railings on the main cornice (12th story), and the railings on the roof between the parapet posts on the three street fronts, are to be made as shown and directed; all to be made of large-sized bar and strap-iron, properly fastened and braced. The railings on the main cornice are to have proper foot-blocks, extending back under the gutter lining to the masonry, to which the posts are to be bolted.

On the coping of the vault wall in rear court, opposite pier F3, provide and put up a railing about 9 ft. long, 8 ft. high, made of 1-in. diameter round bars 6 in. between centres and $\frac{5}{8}$-in. diameter intermediate bars, with 4 $\frac{3}{4}$-in. $\times$ 2 $\frac{1}{4}$-in. cross-bars, put together in the very best manner and securely fastened.

All the windows on second story on all the street fronts are to have on the inside, properly screwed to the wood-work, 2-in. diameter pipe guard-rails with ornamental cast-iron sockets. The surface of the rails and sockets must be finished smooth, and galvanoplated with copper or bronze.

On all the stories pipe rails with sockets are to be provided and put up where directed, in the two shafts between F2 and G2, H2 and I2; in the tank-house pipe rails with proper standards and braces and two rails are to be put up at the edge of the floor.

Gates.—Each of the three entrances on Cortlandt, Church, and Dey streets are to be provided with ornamental gates of wrought-iron, as shown.

Each set of gates is to be made in four folds, and is to have all necessary hinges, bolts, cross-bars, fastenings, and locks put up complete in every respect. The frames are to be made of

heavy bar-iron, and the ornamental strap-work is to be extra heavy.

Guards.—Each of the transom sashes over the main entrance doors on Cortlandt, Church, and Dey streets are to have ornamental guards made of wrought-iron, as shown.

The frames are to be made of heavy bar-iron, and the ornamental strap-work is to be extra heavy; to be properly fastened.

All the windows in the partitions west of the main staircase on the first story, and all stories above, are to have guards made of wrought-iron, frames to be of $\frac{3}{8}$-in. × $\frac{3}{4}$-in. iron filled with twisted lattice-work of $\frac{3}{32}$-in. × $\frac{1}{2}$-in. strap-iron, placed 2 in. between centres, riveted at each intersection and to the frames.

These guards are to be hung on butts and are to have approved fastenings.

Grille-work.—The grille-work for the elevator fronts is to to be provided as hereinafter specified.

Gratings.—On the top of all elevator shafts and the elevator-machine shafts, gratings with trap doors are to be provided; to be made of $\frac{3}{8}$-in. × $1\frac{1}{2}$-in. bars placed $1\frac{1}{4}$ in. between centres, with rods and thimbles, set in frames of $\frac{5}{8}$-in. × $1\frac{3}{4}$-in. iron, properly leaded to the stone copings and fastened to the iron beams and provided with necessary supports. To be properly framed for the elevator ropes and put up complete in every respect.

Gratings made in the same manner are to be put down on the level with the floor on all stories in the shafts between F2 and G2, H2 and I2, as shown.

Gratings are to be provided in the elevator-machine shafts at the cylinder heads.

Partitions, Enclosures, Floors, Etc., made of $\frac{3}{16}$-in. plate-iron, properly fitted and fastened, with all the corners riveted together by means of $2\frac{1}{2}$ × $2\frac{1}{2}$-in. angle-irons, and provided with necessary tee and angle irons for stiffening bars, securely fast-

ened and bolted to the iron-work and masonry, are to be provided and set as follows:

At pier A3 the enclosure of the sidewalk elevator built on an incline is to extend from the sidewalk to the low level basement floor; fastened to the iron floor of the coal vault, between A and A1, A2 and A3, the enclosures under the patent lights are to be placed on an incline, extending from the sidewalk to the basement floor beams.

Between piers B and C the enclosure for the sidewalk elevator is to extend from the sidewalk to the cellar floor; at piers F and G the partitions from the piers to the area walls are to extend from the sidewalk to the patent lights at basement floor level.

Between piers N2 and N3 the enclosures under the patent lights placed on an incline is to extend from the sidewalk to four feet above the basement floor at the inside face of the wall.

The floor of the coal vault on the low level basement floor is to be covered with $\frac{1}{4}$-in. thick plate-iron, with butt joints screwed to cover plates placed under the same and to the flanges of the beams.

The coal chute in the bottom of this floor is to be funnel-shaped, made of $\frac{1}{4}$-in. thick plate-iron with slide cover of approved make, worked by a lever in the cellar.

A chute 20 in. diameter, made of $\frac{1}{4}$-in. thick plate-iron, with the ends cut to fit the batter of the foundation-walls, is to be placed in the cellar between piers A3 and A4, as directed.

The floors of the area at basement floor level between the partitions at piers F and G is to be covered with $\frac{1}{4}$-in. thick plate-iron extending to the inner face of the piers, to have necessary blocking pieces on top of the beams, and tee-iron cross-bars between the beams for supports, to which the plate-iron is to be screwed with butt joints; at pier F a pocket is to be constructed for the sidewalk elevator of the same kind of iron, to allow the platforms to land flush with the floor; the basement floor at the sidewalk elevator between B and C is to

be covered with $\frac{1}{4}$-in. thick plate-iron, three feet wide by full width of opening, properly supported and fastened, and a fascia extending to the bottom of the beam under the same, made of the same kind of iron, is to be provided.

The basement floor at the entrance to the service elevator at F2 is to be covered with a $\frac{1}{4}$-in. thick plate-iron 3 ft. 6 in. wide, forming door-sill, properly fastened to blocking pieces secured to the floor beams.

The bottom of each of the six passenger-elevator shafts is to have a floor made of $\frac{1}{4}$-in. thick plate-iron placed 2 ft. 6 in. below the floor of the ground floor, to have necessary 5-in-beams and tee-irons for supports, to which the plates are to be screwed; necessary holes are to be drilled for the ropes of the elevator machinery.

At the bottom of the service elevator, 2 ft. 6 in. below the basement floor, a floor is to be constructed in the same manner with the four sides 2 ft. 6 in. high, properly fastened and supported; the bottoms of the five elevator-machine shafts are to have wrought-iron pans the full size of the shaft, made of $\frac{1}{4}$-in. plate-iron with sides turned up 6 in. all around, made perfectly water-tight and provided with a flanged outlet for a $1\frac{1}{2}$-in. pipe.

Two additional pans made in the same manner, of sizes as directed, are to be provided and set.

A fascia of $\frac{1}{4}$-in. thick plate-iron is to be provided and set for the outside edge of the tank-house floor, extending from the bottom of the beams to 4 in. above the floor.

Iron Shutters are to be provided for all the windows in the rear walls on all the stories.

All shutters are to have frames made of $\frac{3}{8} \times 1\frac{1}{2}$ in. iron covered with No. 16 crimped iron, properly riveted; to be hung on heavy wrought-iron pin hinges and cast-iron eyes built in. To be fastened with Cornell's patent extension cross-bar or other equally good fixtures, complete in every respect, for securely fastening the shutters when closed and when open.

The shutters of the large windows are to be made in four

folds, and all shutters must fold back when open, close to the walls. In special cases the shutters are to be made as directed.

Iron Doors.—Iron doors are to be made of $\frac{3}{8} \times 1\frac{1}{2}$ in. iron frames, covered with No. 12 crimped iron, properly riveted. All doors are to be hung on heavy wrought-iron hinges, and are to have latch fastenings, except otherwise directed. All doors, except otherwise directed, are to have 3×3 in. angle iron frames for the openings, to be securely fastened.

Iron doors are to be provided in the cellar for openings to the boiler flue stack, to the coal vault and shafts, where shown, to the elevator enclosure between piers *B* and *C*, and in the basement to the same elevator enclosure.

The door to the coal vault is to be made in two sections; the lower section to be hinged to the upper, and provided with proper fastenings.

Posts for Doors.—All doors in fire-proof partitions and sash partitions throughout the building are to have two posts of 4-in. channel-bars, weighing each $8\frac{1}{4}$ lbs. per foot, extending from the top of the floor beams or top of concrete to the bottom of the beams of the floor above.

The tops and bottom are to be connected together by means of $\frac{3}{8} \times 3$ in. bar iron, riveted to the posts, and the upper part of each frame is to be secured to the nearest beam by means of two $\frac{1}{4} \times \frac{1}{2}$ in. straps, which must not project below the bottom of the beams. All channels used for posts are to be perfectly true and straight, out of wind, set perfectly plumb, and to a line.

Each channel-bar is to have holes drilled for fastening the wood-work.

Light Cast-iron Work is to be particularly well made and finished.

All the bases of the show windows on the three streets, comprising the lower portions from the sidewalk to the lower glass-line are to be made of cast-iron, with proper supports. These bases are to have moulded brackets, panels, and cap

mouldings, returns at end and returns extending to the door jambs, all to be properly fitted and fastened to the masonry, the patent lights, steps, and platforms, and must be accurately set to fit the work over the same, which will be furnished under another contract.

All to be made perfectly water-tight. The capping is to be drilled and tapped as directed.

All the store-entrance doors, the outside doors in the basement, and the exterior doors of the roof-house are to have moulded cast-iron saddles.

Deck and Tank House is to be built as shown. The frame is to be made of $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ in. angle-iron placed about three feet apart, with sills and $4 \times 4 \times \frac{1}{2}$ in. angle plates, on which the roof beams rest, all to be well riveted. The exterior of this frame-work is to be covered with No. 12 plate-iron, closely riveted and made perfectly water-tight. On the rear walls the plate-iron is to cover the upper part of the face of the brick wall as shown. The upper part of the plate-iron is to extend in all cases to the top of the angle-iron purlins of the roof. At the door and window frames the plate-iron is to project and is to be screwed to the wood frames.

The cornices and trimmings will be placed on the plate-iron covering under another contract, but the contractor is to do all necessary drilling that may be required for bolt holes.

Patent Lights are to be made of extra heavy frames of cast-iron, rebated and provided with all necessary supports. To be properly fastened and made perfectly water-tight.

The patent lights at the basement floor are to be placed about 3 inches below the floor-line, comprising the bottoms of the area piers C3 and C4, at piers A to B, at piers C to F, at piers G to N, and the roof of the rear vault in court; these lights and the lights over the boiler vault are to be glazed with 3-in. first-quality hexagonal glass, and all the other patent lights flush with the sidewalks are to be made of 2-in. diameter first-quality glass bull's-eyes; the patent lights on Dey

Street, and the parts from piers F to G on Church Street, are to be glazed; all others are not to be glazed, but left open.

All the patent lights are to be set with a proper descent, and those in areas and over the vaults are to be provided with 6-inch long sprouts for 3-inch pipe connections and large heavy 4-in. diameter convex brass stainers, screwed on.

The risers under the show-windows between piers N2 and N3 are to be made of patent lights, glazed with first-quality glass 4 inches square.

The steps, platforms, risers, and cheeks at the entrances to the stores between piers A and A1, F and G, N2 and N3, are to be made with bull's-eye patent lights, glazed as before specified. All other steps and platforms of the other store entrances are to be made of cast-iron, with diamond-pattern tops and nosings, and the risers and cheeks are to be solid with plain panels.

All steps, platforms, and risers are to have all necessary supports, and are to be properly fastened.

All frames of the patent lights are to be made with proper lugs and lips for receiving the work on top of the same.

All the patent lights in the areas and courts are to have an angle-iron flashing, not less than 3 in. high, properly set into the brick-work, forming a water-table made perfectly water-tight.

All the steps to the outside basement doors are to be made of cast-iron, with nosings and risers.

The copings of all the walls in the courts are to be of cast-iron, properly connected with the patent-light flashings and with the walls, made perfectly tight; the shapes of the copings are to be made as directed.

All trap-doors are to be made with heavy bar-iron frames covered with No. 10 plate-iron, properly hung on heavy cast bronze butts and fastened with hasp, staple, and padlock. All are to be provided with proper stays and guard-bars.

The two ventilators over boiler vault and the three ventilators over the vault in the rear court and one ventilator in sidewalk, patent light between piers A2 and A3 (not shown), are to

be made, as directed, of angle-iron frames, covered with No. 10 iron; the tops are to be made same as trap doors, operated by chains from below.

Provide and set where shown on ground-floor plan 24-in. diameter vault covers with cast-iron frames and flanges made of No. 10 wrought-iron extending through the arches. The frames are to be set flush with the flagging, and the covers are to have glass bull's-eyes.

All are to have proper chain fastenings.

Boiler Flue is to be made as shown, and is to be put up in sections as the mason-work is being erected. The boiler flue is to be made with a square box at the bottom, and is to be provided at the bottom with a funnel and 18-in. diameter tube with a trap door. It is to be made three feet four inches in diameter of $\frac{1}{4}$-in. plate iron, riveted together with $\frac{5}{8}$-in. rivets, with a four-inch pitch, made in sections about 20 feet long, and the ends of each section are to have a 4-in. × 4 in. bent angle-irons riveted to the same; the sections are to be bolted together with $\frac{5}{8}$-in. bolts, 6-in. between centres, through the angle-irons.

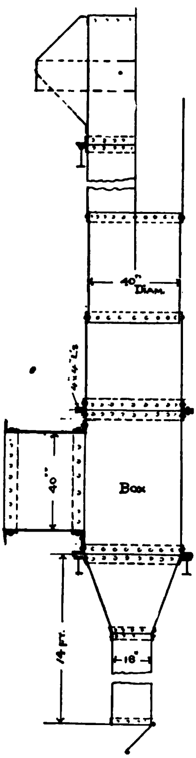


Fig. 61.—Wrought-iron Boiler Flue.

Eeach section is to be supported on two 6-in. beams resting on the walls.

All necessary beams are to be provided at the foot for a proper support.

The horizontal portion is to be made of the same kind of plate-iron, riveted together and to have an angle-iron flanged outlet in the boiler vault. The horizontal section is to extend into the boiler vault as shown, and all necessary supports are to be provided. The top is to have a flange as shown, made of $\frac{1}{4}$-in. thick iron, properly fastened and braced.

Elevator Fronts are to be built as shown on the detail drawings, the pilasters with ornamental fronts and caps and the jambs; the transoms and cornices are to be made of cast-iron. On the ground floor, 1st and 11th stories, the portions above the pilasters are to be panelled and ornamented, and corresponding panels are to be carried across the openings.

The jambs are to be closely fitted to the brick-work.

All the cornices are to have one member ornamented; the various sections are to be made to accurate curves, and the joints must be well fitted.

The soffits are to be carried back the full depth.

Heavy cast-iron ribbed sills extending from jamb to jamb are to be provided for all openings, set flush with the floor. In the shafts the spaces between the soffits and the sills are to be filled with No. 12 crimped iron, accurately fitted; in the ground floor this covering plate is to extend to the iron floor.

All the above work must be securely fastened.

The panels above the transoms are to have panels of wrought-iron grille-work, made of wrought-iron frames to be of $\frac{1}{2}$ in. $\times$ 1 in., and the lattice is to be of $\frac{1}{4}$ in. $\times \frac{3}{4}$ in. bars twisted, riveted at each intersection with strap-work, as shown.

The doors are to be made, as shown, of heavy wrought-iron bars and ornamental strap-work, all properly fastened and riveted. The strap-work of the upper panels is to be twisted, riveted at each intersection.

One of each set of doors is to be stationary and the other door is to be hung on overhead bronze anti-friction sheaves on steel ways, provided with guard-rails, and the lower part is to have proper guides; the doors are to be fastened with extra heavy polished bronze latches of special make and extra strong, striking plates and buffers with rubber heads; all to be put up complete in every respect.

All the grille-work and the doors are to be carefully finished, and are to be Bower-Barffed.

The service elevator located near F2 is to have on all sto-

ries cast-iron sills and cover plates in the shafts from the door-head to the sills, as above specified—all to be properly fastened.

Sidewalk Elevators.—Provide and put up complete in every respect 3 sidewalk elevators—one at A4, one at B, and one at F; the two former are to run from the cellar to the sidewalk, and the latter from the basement floor to the side-walk.

The elevator at A4 is to run on an incline.

All necessary frame-work, guides, gearing, chains, platforms, etc., etc., are to be provided. The winches are to be located where directed. All wood-work is to be of well-seasoned oak. The platforms are to be covered with $\frac{3}{16}$-in. thick plate-iron, properly screwed down. All the elevators are to be delivered in perfect and complete working order.

Miscellaneous.—In all cases where flashings and gutter-linings are to be connected with iron-work, a $\frac{3}{8} \times 1\frac{1}{4}''$ bar is to be screwed or bolted on every 12 inches to the iron-work, clamping the flashings.

Each of the parapet posts on the three fronts is to be provided with $1\frac{1}{2}$-in. thick galvanized cast-iron plate, $20 \times 20''$; each plate is to be bolted down with two 1-in. diam. galvanized iron bolts 6 ft. long, with necessary plates.

The contractor is to remove all refuse materials promptly, and is to do all necessary drilling and cutting of iron-work that may be required and finish up after them.

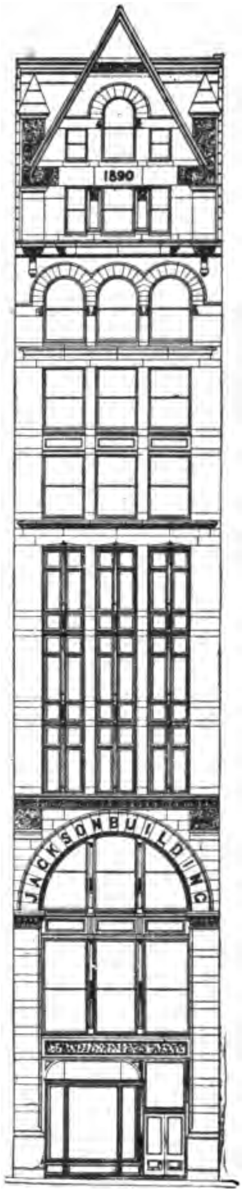


FIG. 62.—FRONT ELEVATION.

CHAPTER VII.

THE JACKSON BUILDING.

THE Jackson Building, situated on the north side of Union Square, New York, on a plot 28.6 × 200 ft., is one of the earliest specimens of the skeleton construction, and embodies all of those features that distinctly pertain to its class. Its foundations are on solid rock, some of the piers supporting the wall columns being as much as thirty feet below the sidewalk.

Its situation on the north side of Union Square, with the Century Building on the east, and the Parrish Building enclosing it on the west, and having an extension through to Eighteenth Street, gives it a commanding character in spite of its narrowness.

It is eleven stories in height, is entirely fireproof, and was finished the first of June, 1892. Both the Union Square and Eighteenth Street fronts are of pink granite from the New England Granite Works, and of buff brick 1½ inches thick. The building extends five stories above the adjoining buildings, the height from the curb level to the top tier of beams being 155 ft. 6 inches.

Iron bases were set November 5, 1891. In one month six stories were erected. All the iron construction to the

seventh story was in place by December 12th. Then the brick-work began to rise rapidly, and on February 12, 1892, the roof tier of beams was set.

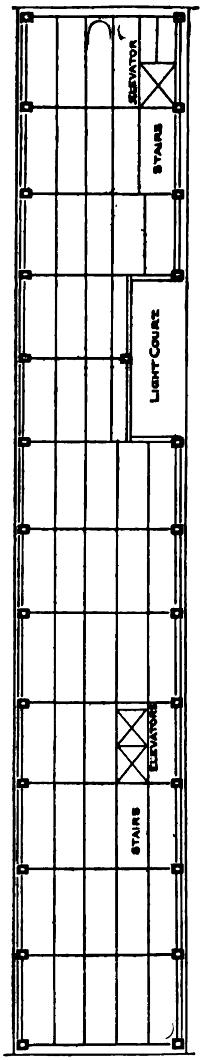


FIG. 63.—TYPICAL FLOOR PLAN, JACKSON BUILDING.

The 1st, 2d, and 3d stories of each front are finished in cast-iron, with neatly moulded cornices and mullions. The 4th, 5th, and 6th stories of the Union Square front have projecting angular copper bay windows; the same stories on Eighteenth Street are finished with a cast-iron bay window, covering the entire front except the side piers.

The stairs are finished with cast-iron ornamental strings and wrought-iron railings with marble treads, the passenger elevators with wrought-iron scroll grilles and cast-iron transoms.

Floor-beam Spacing in the Jackson Building.—In spacing the floor beams and girders in the floor of this building economy of material and simplicity of connection is observed. The cast-iron columns, of which there are twenty-six, are all 12 inches by 16 inches, and set upon cast-iron base-blocks 33″ × 36″ × 1½″ thick by 23 in. in height, arranged in the side-walls as shown on the plan, Fig. 63, placed about 15 ft. apart, standing four inches away from the party lines.

The cross-girders are all 20 inches by 64 pounds per foot I-beams, while at right angles to the same are placed the 9″ × 21 lbs. per foot floor beams secured to the girders by wrought-iron knees and bolts, as shown at Fig. 64, and spaced about 5 ft. 3 inches apart.

A twelve-inch curtain wall extends between the columns from the bottom of basement to the top of sixth story, supported at each tier of beams by two I-beams sufficient in strength to carry this wall. At the top of sixth story the columns cease and a 20-in. brick wall begins, built upon beam girders encircling the entire building, resting upon the top of columns and thoroughly anchored to the 7th-story floor girders. The 20-inch wall extends through three stories,—that is, the

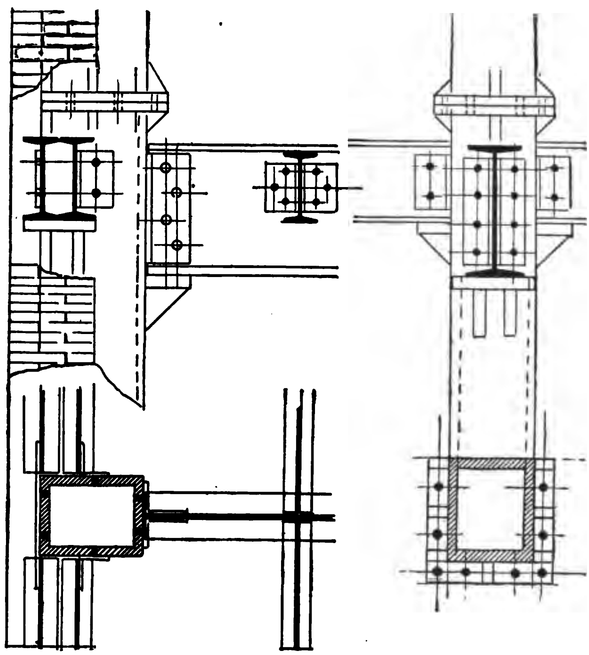


FIG. 64

seventh, eighth, and ninth,—and then a 16-inch wall extending through the tenth and eleventh stories completes the building. The floor beams of the eighth, ninth, tenth, and eleventh stories are 15 inch by 41 pounds per foot, spaced about 5 feet from centre to centre, and thoroughly anchored to the brick walls.

Calculation of Floor Weights.—The different materials calculated as dead load were: Fireproof floor arches, beams, wooden flooring, filling above arches, fireproof partitions plastering upon ceiling, and plastering upon partitions, when added together was found to be 103 *pounds per square foot.*

The live load assumed was 50 pounds per square foot, making a total of 153 pounds.

The wall weights were 153 pounds for the 16-inch wall, and 192 pounds for the 20-inch wall. The weight of columns was also added to the total weight. The following table gives the total loads on the columns of each story:

	Length.	Size.	Load.
6th-story columns	12 ft. 6 in.	12″ × 16″ × 1″	177 tons
5th " "	12 ft. 6 in.	" "	195 "
4th " "	13 ft. 9 in.	" × 1¼	213 "
3d " "	15 ft. 7½ in.	" × 1½	231 "
2d " "	16 ft. 10½ in.	" × 1½	250 "
1st " "	17 ft. 6 in.	" × 1½	270 "
Basement "	12 ft. 6 in.	" × 1½	290 "

Column Connection.—In joining the columns with each other, with the floor girders and the curtain-wall girders, simplicity of design was again considered; also a rigid connection. By referring to the detail, Fig. 64, it will be noticed that the 20-inch floor girder rests upon a heavy bracket about 2 inches thick, projecting 6 inches, and that lugs are entirely dispensed with. The girders were all made of one length, with only ⅛ of an inch clearance; 6″ × 4″ × ½″ angle-knees were riveted to each side, projecting slightly beyond the end, so that the columns and girders were drawn perfectly tight.

The girders supporting the curtain wall are also shown in this figure, the same principle being carried out as for the floor girders. In addition, a 6″ × ⅝″ wrought-iron strap is bolted to the column, and helps tie these girders to each other and to the column.

By referring to Fig. 65, another method of securing a rigid connection with the floor girders of a building seems practi-

cable—two I-beams being used in the place of a deeper one. Where one beam is used considerable furring is required, but in a two-beam girder the ceiling can be made perfectly level.

The curtain-wall girders can be accommodated in the same

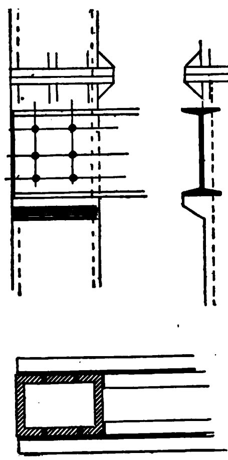


FIG. 65.—DOUBLE-BEAM GIRDER CONNECTION WITH CAST COLUMNS.

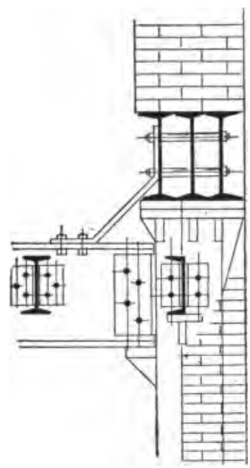


FIG. 66.—DETAIL OF TOP OF COLUMN, SHOWING GIRDERS SUPPORTING UPPER WALLS.

manner as shown in Fig. 64, but larger knees and more bolts will be required.

In resting one column upon another, eight bolts $\frac{3}{4}$ of an inch in diameter are used, and the flanges are stiffened by small brackets cast with the column, as shown in the figure.

The beam girder, as previously mentioned for the support of the walls enclosing the upper stories, is shown in detail, Fig. 66. Three 20 I-beams were used, cased upon the outside by a $\frac{1}{4}$-in. thick cast-iron plate; the girder being secured by bolts to the top of columns, and further secured by a $6'' \times \frac{1}{2}''$ strap to each floor girder. The curtain wall is carried up to the bottom of the three-beam girder, while the floor arch is supported by a channel on the same level as the floor beam.

CHAPTER VIII.

THE NEW NETHERLAND, NEW YORK.

THE plans as prepared and carried out under the direction of Mr. Wm. H. Hume, architect, complete a building which in some respects is interesting and imposing. No finer site could have been chosen for such a structure, standing as it does at the portal of New York's great Park, and towering far above any of the tall buildings for which this locality is noted.

The building covers four city lots, with a frontage on Fifth Avenue of 100 feet, and a depth on Fifty-ninth Street of 125 feet, with a cellar and basement below the sidewalk, and seventeen stories above; the four upper stories of which are in the angle or slope of the roof, thus somewhat reducing the height of the structure.

To complete the building it required nineteen tiers of steel beams and girders. The height from sidewalk to top tier of beams is 216 feet, with an additional 18 feet to top of roof houses, making the total height 234 feet, and as previously mentioned, nine hundred steel columns and about forty-five hundred steel beams were used in the construction.

The style of the building is of Modern Romanesque design. The first four stories are built of heavy rock-faced Belleville brown stone, thus affording a strong and massive base; the superstructure is of buff brick, relieved with stone and terra-cotta trimmings. The twelfth story is faced entirely with a heavy cornice finished by a balcony and stone balustrade, the whole story forming the main cornice of the building, and so

arranged as to break in the most pleasing manner the towering appearance of the building. In construction, it is as thoroughly fireproof as it is possible to make it, while in strength it is only necessary to state that the brick walls are relieved of the

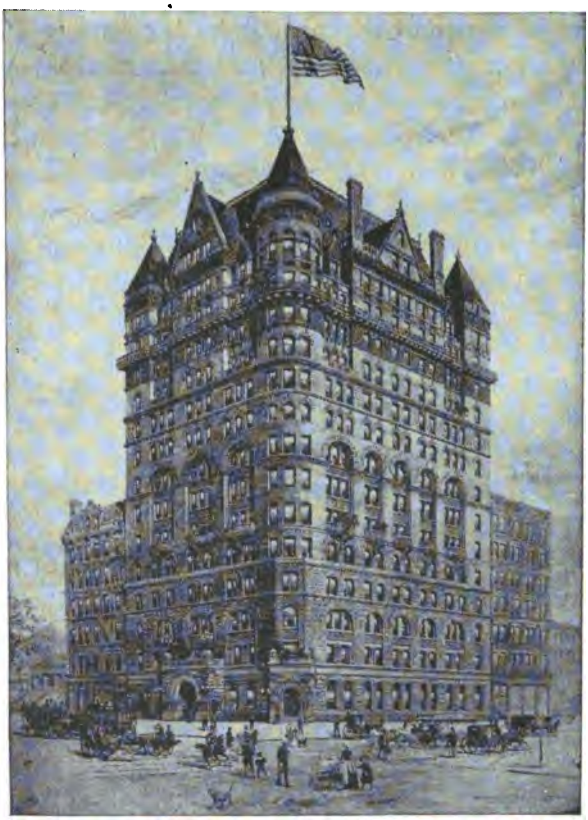


FIG. 67.—THE NEW NETHERLAND, FIFTH AVENUE, CENTRAL PARK ANI FIFTY-NINTH STREET.

strain and weight imposed by the use of heavy steel box columns, made of plates and angles.

The main office, covered by a dome of iron and glass, and finished with bronze, forms a notable feature.

The grand staircase is of marble and bronze, supported upon heavy steel I-beam strings.

Marble and bronze are also extensively used in good taste throughout the main hall, office, and various other rooms on the ground floor.

The continuation of the main stairs is constructed of cast-iron strings, cast-iron ornamental risers with marble treads and guarded by ornamental wrought-iron railings. The several passenger-elevators fronts are finished and constructed with cast and wrought iron. The main passenger elevators on the ground floor are finished in bronze.

The roof is constructed of 7-inch steel-beam rafters, about 4 feet apart, fitted to the outside floor beams of the different stories in the inclined portion, supporting $3'' \times 3'' \times \frac{1}{2}$ T's as purlins, placed horizontally 25 inches apart. The dormers are made of cast-iron, to which are secured terra-cotta blocks and copper flashings. The entire pitched roof is covered with terra-cotta blocks and tiles.

Over all doors and window-openings in brick walls are placed cast-iron lintels, with one or more webs to suit the thickness of the walls, and of sufficient thickness of metal to carry the imposed loads; in all the openings in the fireproof partitions light T's were used.

The ceilings of halls, corridors, bath rooms and closets were furred down with light T's spaced about $16\frac{1}{2}$ inches from centre to centre, to carry terra-cotta blocks.

The space between these hanging ceilings and floors is used as vent flues, to carry the vitiated air from closets, etc., through and out the roof.

Floor Plan.—The floor plan, Fig. 68, shows the manner of dividing the floor area into rooms, halls, closets, elevators, stairways, etc., to the best advantage. Each apartment has its parlor, bed room, bath room, and closets, with a separate air-shaft from the bath room. These air-shafts extend to the roof and topped out upon the roof with a small house con-

structed of T and angles, covered on the sides with iron, and on the top with glass and iron. In the sides of these houses are electric motors and fans.

The various rooms throughout the building are all light; those on the two fronts face Fifty-ninth Street and Fifth Avenue, those on the inside the large court in the centre of the building. At the bottom of this centre court the dome skylight is situated, covering the entire office and grand stairway.

At the extremity of the hall to the left is the ladies' eleva-

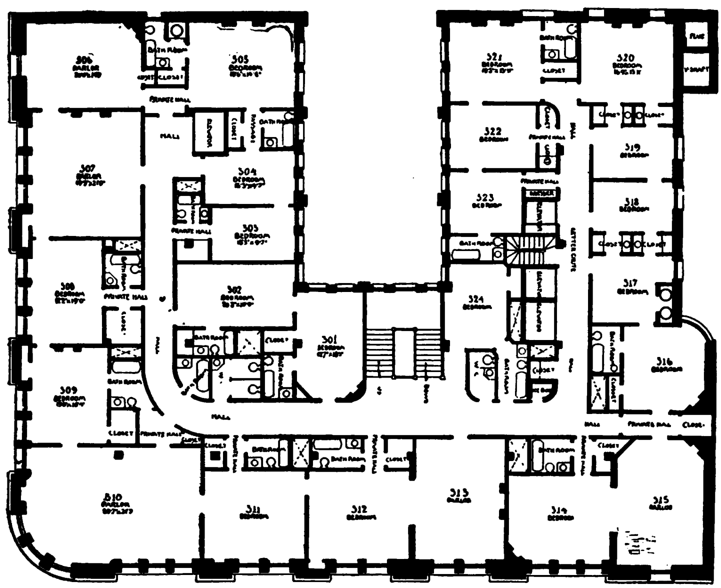


FIG. 68.—TYPICAL FLOOR PLAN.

tor; at the extreme right is the main elevators, with an electric elevator for freight, etc., adjoining the main elevators and servants' stairway. The servants' stairway throughout the entire height of building is constructed of cast-iron strings and risers and slate treads.

The main boiler flue, built of brick, is situated at the north.

east corner immediately adjoining the large vent shaft. This flue and shaft is thoroughly tied to the building, at each tier of beams, by heavy wrought frames and anchors.

Beam Plan.—The beams and girders of the building are arranged in a systematic and economical manner upon the

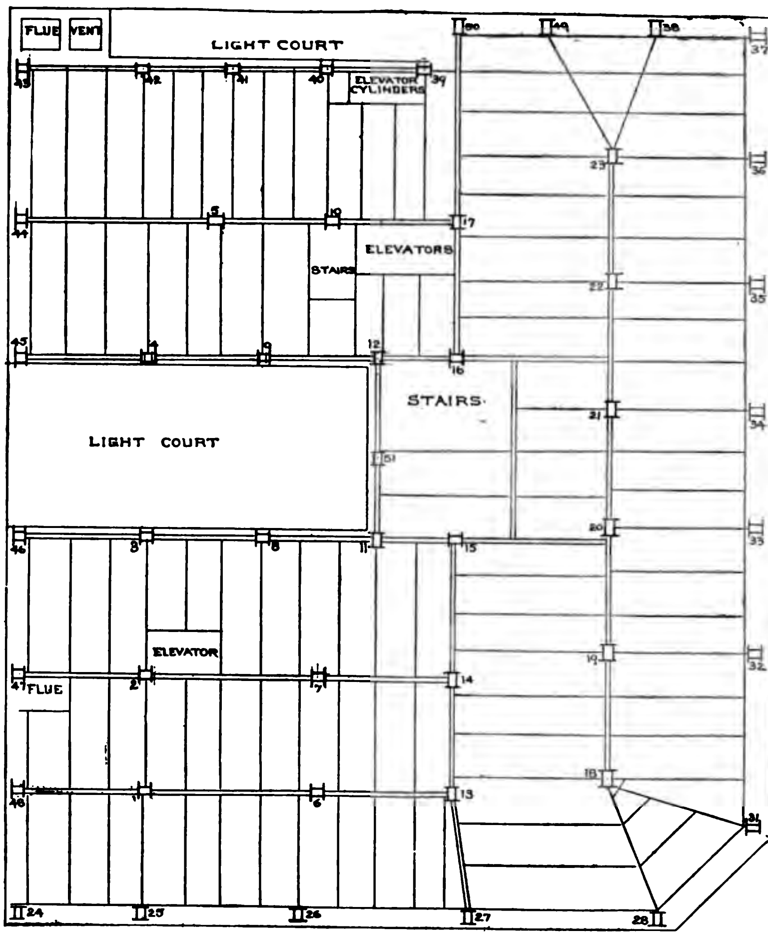


FIG. 69.

plan, making a strong and rigid structure throughout. By referring to the plan, Fig. 69, the girders are shown extending

in different directions, and in the majority of the spans are composed of two 15-inch, 32 lbs. per foot channels placed with the flanges toward each other, thus reducing to a considerable extent the cost of framing of the beams.

When by the arrangement of the floor beams a floor arch is to be supported, the flange of the channel is turned toward the arch—such, for instance, as that shown between columns marked 27, 13, 14, and 15. When two channels were not sufficient to support the floor weight, two 15-inch by 75 lbs. per foot I-beams were used (see spans between columns 17 and 50, 5 and 44).

The weight calculated upon beams, girders, and columns is 175 pounds per square foot of surface, which includes the total dead and live load.

The columns are spaced from 15 to 17 feet apart, except the distance between columns 17 and 50, which is 27 ft. 2 inches.

The openings throughout the floors for stairs, elevators, and flues are framed with beams and channels, as shown upon the plan.

Columns 49 and 38 are placed in the position shown to equalize the distance between 37 and 50, an arrangement which serves to brace the end wall by placing the beam girders between the columns 23, 49, and 38 on a skew. This same principle is also carried out on the opposite end of the building between columns 18, 28, and 31.

The column marked 51 at end of light-court adjoining the stairs is supported by a plate girder, which in turn rests upon brackets, secured to columns marked 11 and 12.

The floor beams throughout the building are 12-inch by 32 lbs. per foot (except between columns 17 and 50 where they were made 15-inch by 41 lbs. per foot), and stiffened by $\frac{3}{4}$-inch diameter tie-rods spaced on an average of 5 feet centre.

Columns.—The details of the columns and the column connections in this building are, without exception, the best that could be designed for such a structure.

By referring to the detail of column, Fig. 70, it will be observed that the girders (of two 15-in. beams in this case) are

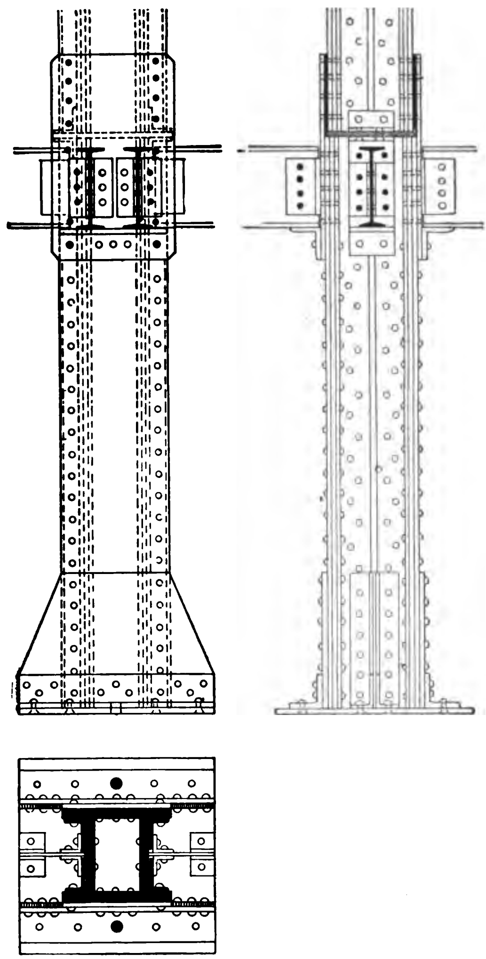


FIG. 70.—DETAIL OF CELLAR COLUMN.

secured to the columns by twelve rivets through angle-knees 4 in. × 4 in. × ½ in. thick.

The inside knees are first riveted to the column at the shop

and the outside knees are left loose, so the beams can be placed in position; then the entire work is riveted together by hot rivets at the building.

The same connection also applies to the floor beams and girders joining the columns.

Angle-knees are also riveted to the column at the heel of the beams and girders, as shown in the figure. In securing the columns to each other heavy steel planed plates were placed between at about the floor levels, so as to allow a proper clearance for the girders; then each side of the columns were placed, similar plates covering the joint, and extending at least two feet above and below the joint; and when the immediate column above is less in size than the one below filler plates are used, shown by the heavy black line, and riveted through by the same sized rivet as used in the body of the columns.

Rivets of 1-in., $\frac{7}{8}$-in., and $\frac{3}{4}$-in. were used throughout the building; 1-in. in the heavier and $\frac{3}{4}$-in. in the lighter columns.

The above detail also applies to the Z-bar column.

Foundations for Columns.—To prepare the foundation for the weight to be supported the rock was cut away to a depth of three or four feet, so there could be no chance of any decayed portion remaining; then several blocks of hard granite were dressed and set upon each other at about an angle of 30 degrees. On top of this a heavy steel plate 2 in. thick was bedded, and heavy anchors $1\frac{7}{8}$ in. in diameter were built extending down through the granite, passed up through the 2-in. plate and foot of column, and held securely by heavy nuts.

For crushing strength of stone, etc., see chapter on Foundation.

Where buildings of this weight and height are built upon rock very much expense is saved, many calculations are dispensed with, and the danger of any settlement is reduced to a minimum.

STEEL COLUMNS—NEW NETHERLANDS.

Column Marked No. 17.

	Length.		Outside Plates.		Webs.		Angles.		Load, Tons.
	Ft.	In.	No.	Size in In.	No.	Size in In.	No.	Size in In.	
Cellar	12	9	6	$20\times\frac{13}{16}$	4	$13\times\frac{7}{8}$	4	$6\times4\times\frac{13}{16}$	855
Basement	12	9	"	$18\times\frac{13}{16}$	"	"	"	"	808
1st story......	17	6	"	$16\times\frac{13}{16}$	"	"	"	"	763
2d "	13	"	"	$16\times\frac{3}{4}$	"	"	"	"	718
3d "	12	"	4	$16\times\frac{7}{8}$	"	"	"	"	673
4th "	"	"	"	$16\times\frac{3}{4}$	"	"	"	"	628
5th "	"	"	"	$16\times\frac{5}{8}$	"	"	"	"	583
6th "	"	"	"	"	"	$13\times\frac{3}{4}$	"	"	539
7th "	"	"	"	"	"	$13\times\frac{5}{8}$	"	"	494
8th "	"	0	"	$16\times\frac{9}{16}$	2	$13\times\frac{7}{8}$	"	"	449
							z	bars.	
9th "	"	"	2	$16\times\frac{7}{8}$	1	$8\times\frac{7}{8}$	4	$6\frac{1}{8}\times3\frac{5}{8}\times\frac{7}{8}$	404
10th "	"	"	"	$16\times\frac{11}{16}$	"	"	"	"	359
11th "	"	"	"	$15\times\frac{1}{2}$	"	"	"	"	314
12th "	"	"	none	....	"	"	"	"	270
13th "	11	6	"	....	"	"	"	$6\frac{1}{16}\times3\frac{9}{16}\times\frac{13}{16}$	225
14th "	12	3	"	....	"	$8\times\frac{11}{16}$	"	$6\frac{1}{8}\times3\frac{5}{8}\times\frac{11}{16}$	190
15th "	12	0	"	....	"	$7\times\frac{1}{2}$	"	$5\frac{1}{16}\times3\frac{5}{16}\times\frac{1}{2}$	135
16th "	11	6	"	....	"	$6\times\frac{7}{16}$	"	$4\times3\frac{1}{16}\times\frac{7}{16}$	90
17th "	13	6	"	....	"	$5\frac{1}{4}\times\frac{1}{4}$	"	$3\times2\frac{1}{4}\times\frac{1}{4}$	45

Columns Marked 25, 28, 31, 32, 33, 34, 35, 36, 38, 45, 46, 49, 50.

							an	gles.	
Cellar.........	12	9	4	$20\times\frac{7}{8}$	2	$13\times\frac{7}{8}$	4	$6\times4\times\frac{13}{16}$	630
Basement.....	12	9	"	$20\times\frac{13}{16}$	"	"	"	"	593
1st story......	17	6	"	$20\times\frac{3}{4}$	"	"	"	"	558
2d "	13	"	"	$18\times\frac{3}{4}$	"	"	"	"	518
3d "	12	"	"	$16\times\frac{3}{4}$	"	"	"	"	489
4th "	"	"	"	$16\times\frac{5}{8}$	"	"	"	"	453
5th "	"	"	"	$16\times\frac{1}{2}$	"	"	"	"	419
							z	bars.	
6th "	"	"	2	$16\times\frac{7}{8}$	1	$8\times\frac{7}{8}$	4	$6\frac{1}{8}\times3\frac{5}{8}\times\frac{7}{8}$	384
7th "	"	"	"	$16\times\frac{11}{16}$	"	"	"	"	349
8th "	12	0	"	$16\times\frac{1}{2}$	"	"	"	"	314
9th "	"	"	"	"	"	$8\times\frac{3}{4}$	"	$6\times3\frac{1}{2}\times\frac{3}{4}$	280
10th "	"	"	none	....	"	$8\times\frac{7}{8}$	"	$6\frac{1}{8}\times3\frac{5}{8}\times\frac{7}{8}$	245
11th "	"	"	"	....	"	$8\times\frac{3}{4}$	"	$6\times3\frac{1}{2}\times\frac{3}{4}$	210
12th "	"	"	"	....	"	$8\times\frac{5}{8}$	"	$6\frac{1}{16}\times3\frac{9}{16}\times\frac{5}{8}$	175
13th "	11	6	"	....	"	$8\times\frac{1}{2}$	"	$6\frac{1}{8}\times3\frac{5}{8}\times\frac{1}{2}$	138
14th "	12	3	"	....	"	$8\times\frac{5}{8}$	"	$6\times3\frac{1}{2}\times\frac{5}{8}$	105

Wall Thicknesses.—The thickness of the front walls facing Fifth Avenue and Fifty-ninth Street are, for the cellar 3 ft. 4 in.; basement, 3 ft.; first story, 2 ft. 8 in.; second, third, fourth, and fifth stories, 2 ft.; sixth, seventh, and eighth, 1 ft. 8 in.; ninth to and including fourteenth story, 1 ft. 6 in. The fifteenth, sixteenth, and seventeenth stories are in the inclination of the roof.

The walls of the light-court are all twelve inches in thickness and supported by the channel-girder and cast-iron plate, as shown in the plan and section of the wall, Fig. 71. The

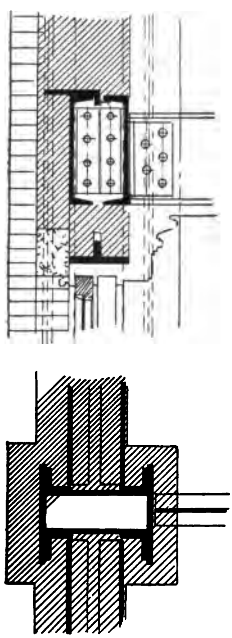


FIG. 71.

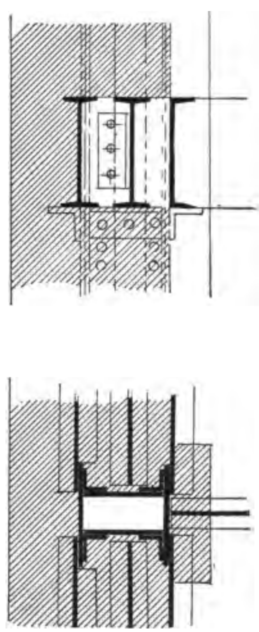


FIG. 72.

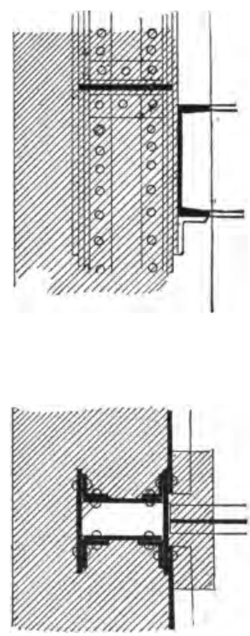


FIG. 73.

columns, as well as the face of girders in this light-court wall, are faced with 4-in. enamelled brick.

Over the head of each window, supporting the entire thickness of wall, is a stone and cast-iron lintel, and at the bottom of windows a stone sill is used. The columns are also encased on the inside by a 4-in facing of terra-cotta blocks. This entire

construction enclosing the court starts from the level of the second story, and extends to the roof or to the top of seventeenth story and topped out by a blue-stone coping.

The front walls, facing on Fifty-ninth Street and Fifth Avenue from the cellar to the eighth story, are shown by the plan and section Fig. 73. The channel extending along the wall is the floor channel, which receives the floor arches. Fig. 72 represents the plan and section of walls above the eighth story; the beams and outside channel support the wall, while the inside channel supports the floor arches.

Hotel Waldorf.

The Hotel Waldorf, which Henry J. Hardenbergh, architect, has planned and built for William Waldorf Astor, situated at the corner of Thirty-third Street and Fifth Avenue, New York, covers a plot of ground two hundred and forty-nine feet six inches (249 ft. 6 in.) on Thirty-third Street, and ninety-eight feet nine inches (98 ft. 9 in.) on Fifth Avenue. The construction is entirely fireproof, and a variation of the skeleton construction as heretofore described. The walls simply carry their own weight, while the floors and their loads are supported upon cast-iron columns built in with the masonry. The constructive work is protected by terra-cotta and other fireproof material.

The style of the building is in the German renaissance of the sixteenth century. The first two stories of the front are built of heavy blocks of brown stone; the third, to and including the ninth, of red pressed brick, trimmed with elaborately carved brown stone and terra-cotta, while the three upper stories are in the angle of the picturesque roof of gables and towers—a total of twelve stories above the sidewalk.

Floor Plan.—The plan Fig. 75 shows a typical floor of the building, divided into light-courts, halls, stairways, and rooms. The east court, nearest Fifth Avenue, is 13 ft. wide by 65 ft. long; the extreme west court is 13 ft. wide by 58 ft.

long, and the centre or garden court is 40 by 50 ft. Skylights of iron, glass, and copper cover the first story at the bottom of the east and west court, while the first story of the garden court is covered with a revolving dome skylight 107 ft. in circumference, constructed of cast-iron, glass, and copper.

The main stairway, opposite the three elevators, and the

FIG. 74.—THE WALDORF.

stairway from the long hall, are constructed of cast-iron strings, cast-iron risers, and white marble treads, guarded by ornamental wrought-iron and cast-iron railings. The elevator fronts are also constructed of the same material, and enclosed

on the front with exquisitely wrought grille-work; the lower stories have the jambs of the elevator enclosure covered with onyx and glass.

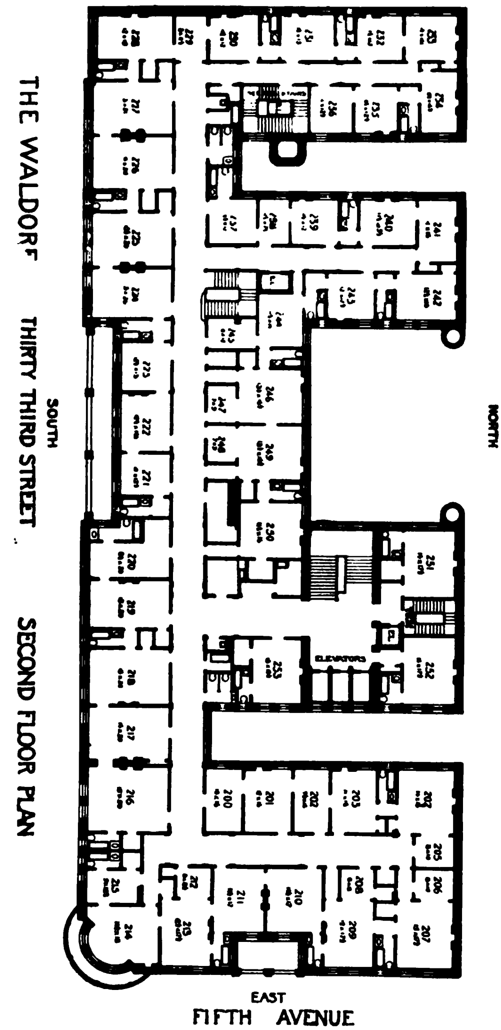


FIG. 75.—TYPICAL FLOOR PLAN.

The elevator enclosure of the west side is constructed of

wire-work and angle-iron, surrounded by a staircase built of cast-iron strings, cast risers, and slate treads.

Beam Plan.—On account of the construction being similar throughout, only that portion of the building is shown

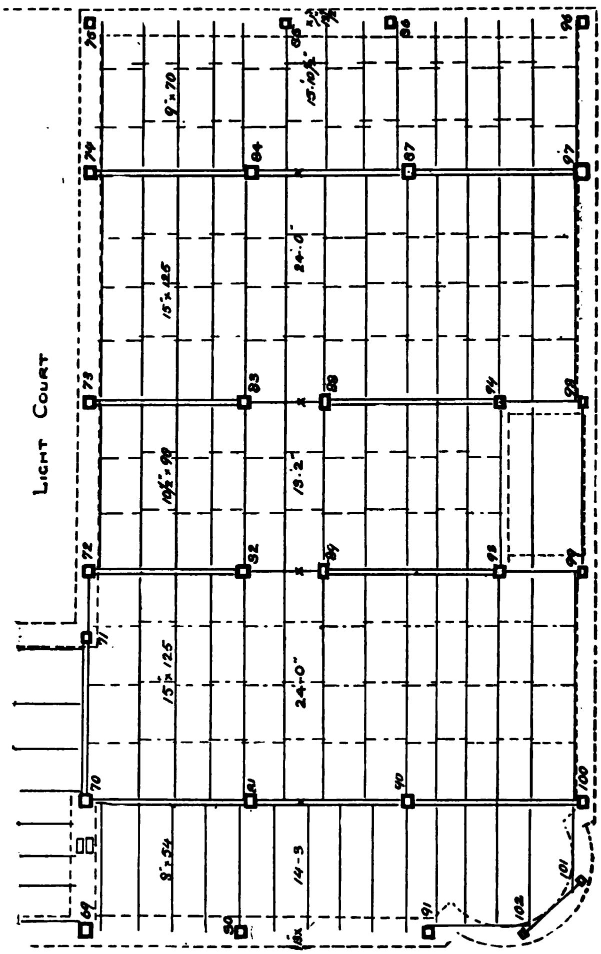


FIG. 76.—BEAM PLAN AT FIFTH AVENUE END.

bounded by Fifth Avenue and the east light-court; the plan represents the arrangement of the columns, girders, and beams above the first story.

The columns are so arranged upon the plan, Fig. 76, to not interfere with the planning of the rooms; the beams and girders being spaced to support 175 lbs. per square foot of floor surface, which includes the total live and dead load. The cross-girders are two 15″ × 125 lbs., and two 15″ × 150 lbs. per yard I-beams, depending upon the span; the beams of the 24.0 span are 15″ × 125 lbs. per yard, of the 19.2 span 10½″ × 90 lbs. per yard, with smaller beams in the shorter spans, all spaced from 3 ft. 6 in. to 4 ft. 4 in. centres.

The columns range from 16″ × 16″ × 1½″ in the basement to 7″ × 7″ × ¾″ in the roof. The connections with the floor beams and girders are made similar to the detail of cast-iron columns with iron girders under chapter on Column Connections. The outer or wall columns of this portion of the plan, as well as those throughout the building, extend through the entire height of the masonry, resting upon cast-iron foot-blocks and rock bottom. The inside columns, or those marked 81 to 84, 87 to 90, 93 and 94, are all supported upon heavy double box girders in the ceiling over the large dining-room of the first story; these girders are four in number, 4 ft. in depth at the centre, 32 in. in width, and 36 to 38 ft. in length, and supported upon large cast-iron columns 16 in. in diameter.

The slope of the entire roof is constructed of 6-in. light I-beam rafters, placed about 4 ft. centres and covered with 3″× 3″× T's 25 in. centres supporting porous roofing-blocks; then finished off with English roofing-tile. The entire construction of the gables and towers is similar to the main roof.

THE POSTAL TELEGRAPH BUILDING.

The Postal Telegraph Building, as designed by George Edward Harding and Gooch, architects, at the corner of Murray Street and Broadway, New York, is a fireproof structure fourteen stories in height, with a sub-basement and cellar below the sidewalk. The constructive material is of *cast-iron*

columns and *steel* floor beams and girders throughout. Above the sixth story the walls are carried on steel girders, thus economizing the floor space on the lower stories.

The entrance is 30 feet wide, semicircular in shape, and from it the doors leading to the main hall, store, messenger,

FIG. 77.—THE POSTAL TELEGRAPH BUILDING, NEW YORK.

and despatch rooms. This circular entrance is trimmed largely with choice marbles, with which material the main hall, as well as all the halls, are wainscoted.

Mosaic tiling is employed in the hallways and in other

prominent places throughout the building, which aid in making it one of the attractive and complete structures in Broadway.

The entrance is flanked by massive piers projecting from the main walls, and capped by bas-reliefs representing light and electricity.

Indiana limestone effectively carved and wrought is carried up four stories; above the fourth story the building is finished in light gray brick, with terra-cotta ornamentation.

All partitions are constructed of terra-cotta or fireproof blocks; the fireproof floor arches are covered with a smooth surface of Portland cement.

Iron stairways with marble steps extend from the main floor to roof, and from large passenger elevators give access to all the floors, while two express elevators are used exclusively for the four upper stories.

The building covers a plot of ground 70 feet 2⅜ in. on Broadway by 155 feet 6½ in. on Murray Street, with an extension at the west end north from Murray Street.

The beams and girders are arranged to support 175 lbs. per square foot of floor surface, the beams being 15 in. by 41 and 12 in. by 32 lbs. per foot spaced from 4 to 4 feet 6 inches centre.

The construction of the column joints and beam connections are similar to those of the Waldorf.

CHAPTER IX.

WIND-BRACING.

THE subject of wind-bracing is receiving considerable attention at the present time among those directly interested in the designing and constructing of high and narrow buildings.

FIG. 78.—VENETIAN BUILDING, CHICAGO, ILL.

Very many criticisms by engineers have appeared from time to time in the weekly and monthly periodicals; but we have failed to see any system, with one or two exceptions, proposed that would meet the full requirements of architects.

It is no doubt a difficult problem at the least, and whether lateral bracing is adopted will depend in a great measure upon examples of buildings which have been previously built, and which seem to be perfectly secure from all lateral displacement.

In using columns and girders made up of plates and angles with *knee-braces*, as those shown under chapter on Column Connections, a great amount of rigidity is secured, and these connections will serve in the majority of cases where the regular transverse bracing would interfere with the necessary openings in the partition and otherwise with the planning of the structure.

The action of the wind against the side of a building produces the effects of overturning and shear, both greatest at the highest point of external resistance, which is the roof of adjoining building, if there be any, or otherwise the surface of the ground. The overturning or the lift on the windward side is likely always to be less than the resistance of dead weight; but the shear is liable to be overlooked, and is probably the cause of the collapse of most of the buildings destroyed by wind, especially during construction, while the walls are newly set.

Wind-pressure.—Experimenters upon the subject of wind-pressure assume that the horizontal pressure of wind against an inclined surface, as a roof, is about 1 lb. per square foot per degree of inclination to the horizontal. For example, if the roof has an inclination of 30 degrees with the horizontal, the pressure of the wind will be about 30 lbs. per square foot of surface. Roofs are generally designed for pressures averaging about 40 lbs. per square foot, but the sides upon which the roof rests for little or none.

The experiments of the Forth bridge engineers, and also other experiments, show conclusively that the pressure per unit of surface is less over a large area than over a small one, and what intensity of wind-pressure it is proper to assume upon a high building is an important question to settle. We are well aware that wind develops considerable energy at times, and we cannot expect to resist its utmost power in the designing of the structure; but we can at least estimate for high velocities of wind, say from 30 to 50 lbs. per square foot, and low intensities of strain in the material.

Wind-bracing in the Venetian Building, Chicago, Ill.—An article by C. T. Purdy, C.E., in the *Engineering News*, of December, 1891, describes in detail the wind-bracing used in the Venetian Building, Chicago.

The Venetian Building is probably as well braced and

its bracing as well disposed as any building using a system of lateral braces.

Fig. 79 is a diagram of the floors of this building, showing the arrangement of the columns and position of the bracing.

Fig. 80 shows the position of the struts and diagonals and the position they occupy in relation to the floors.

The struts of the first and second stories are 1 foot 9 in. below the next story above, and those above slightly less.

Fig. 81 is a strain sheet for the wind-bracing, excepting only the column strains, which were included in the schedule given for the vertical loads on the columns.

The dead weight of the floors is taken at 100 lbs. per square foot. The live load on the first floor 80 lbs., and floors above 60 lbs. The whole of the dead load and about one half the live load is carried into the columns.

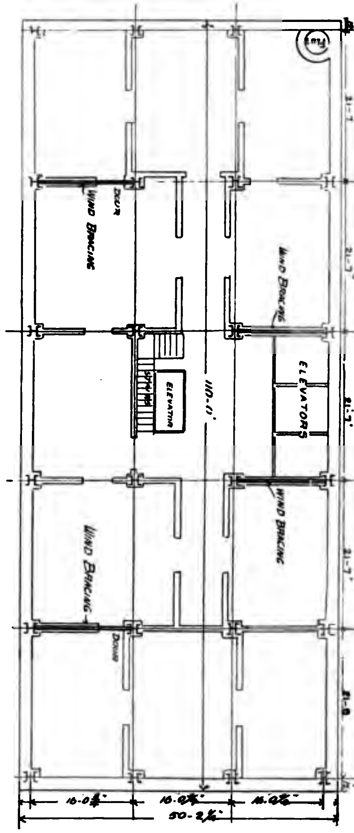


FIG. 79.—TYPICAL FLOOR PLAN OF VENETIAN BUILDING, CHICAGO, ILL.

The calculations of the strains given on the diagram were made as follows: Each set of bracing was figured to resist the wind-force for an area equal to half the height of a story and half the height of the next one above by 21 ft. 7 in. multiplied by 40 lbs., the calculated wind-pressure per square foot of surface.

The total shear at any of these points—*that is, at any floor*

level—is equal to the sums of the shears acting directly on the points above it. It has not been deemed necessary, however, to carry the whole amount of this shear into the bracing, as in any building the dead weight of the structure itself acts to some extent to counteract the distorting effect due to lateral force.

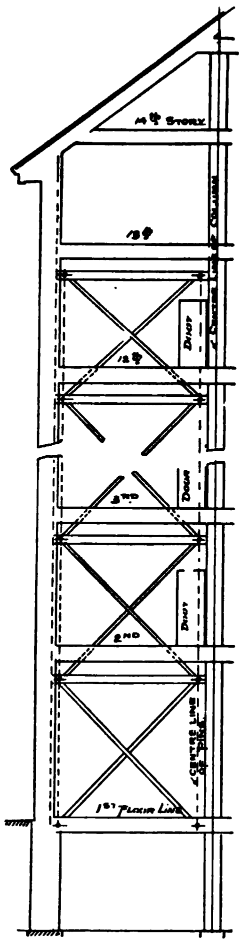


FIG. 80.

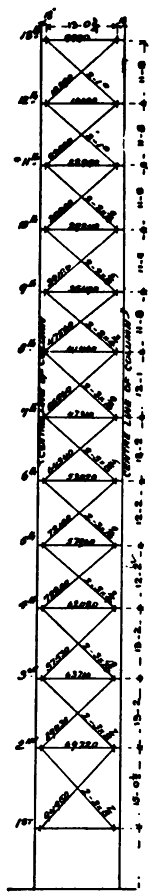


FIG. 81.

PART TRANSVERSE SECTION AND WIND-STRAIN DIAGRAM OF THE VENETIAN BUILDING.

These shears are reduced to some extent on this account. The bracing is then made to resist 70 per cent of the wind-pressure.

All the columns affected by this bracing have been made continuous from the basement to the second floor.

In the cases where the rods come down to the first floor level the bottom strut is connected to the columns so as to take both tension and compression horizontally, as well as to resist the vertical component of the rod strain. This insures the resistance of both columns to the horizontal thrust of the strut, whichever pair of rods is strained, and the columns are calculated to resist the bending moment incurred, as well as to carry their regular column load.

The horizontal struts from the first to the eighth floor are made of two nine-inch channels, arranged somewhat similar to those shown on the Havemeyer Building; flat latticing $2\frac{1}{4} \times \frac{1}{4}$ in., being used top and bottom in place of a plate.

Above the eighth floor lighter channels were used.

The struts are reinforced at the pier points to resist the bending moment of the strut caused by moving the pier centre so far from the centre of the column.

The diagonal steel rods are all dimensioned for 20,000 lbs. unit strain, and no rod is less than $\frac{7}{8}$ inch square.

All these rods are provided with turn-buckles.

The channel struts are so arranged between the columns that the rods pass each side of the column girders, as shown on Fig. 80. By this arrangement they do not interfere with the door-openings in the partitions.

There is but slight connection made to these columns by the horizontal struts. The struts are planed at both ends and no clearance is allowed for connection, so that they have butting joints to the columns. Open holes are provided for four rivets connecting the columns, but these are hardly necessary.

Underneath the end of the strut is a solid cast-iron block, and underneath the block are two bracket-angles, secured to the column with sufficient rivet area to resist the vertical component of the rods in this direction. Above the end of the strut is another cast-iron block, planed on top and bottom to fit in tightly between the strut and the cap plate of the column. This block is made to fit the recess made by the flanges of the Z-bars so closely that the $\frac{3}{4}$-inch cap plate is brought into direct shear entirely around three sides of the block. The shear resistance of the plate together with the weight of the beam directly upon it are more than enough to resist the upward vertical component of the rods. The use of cast-iron blocks in this connection has been found very convenient, for it often occurs that the bracket angles cannot be

brought directly under the channels of the strut, and the medium between the strut and the bracket angles must act as a beam as well as a filler.

From the above system of bracing we find that *every weight caused by the horizontal wind-pressure against a building is transmitted through its own system of triangles to the base or foundation.*

The load on any brace is equal to the sum of all the weights upon its system between it and the upper portion or unsupported end of the building.

In the majority of cases the wind-pressure need not be considered below the fifth or sixth story, this being the average height of adjoining buildings.

CURTAIN WALLS.

Section 485 of the New York Building Law, given in Chapter I of this volume, describes the thicknesses and manner of supporting the curtain walls of the skeleton frame, and will not be repeated. In making a comparison between that required by the skeleton frame and the old method we find that considerable space is gained on the inside measurements of the building. In the ordinary method, by the same law, in an example of a warehouse, store, factory of, say, twelve stories (150 feet in height):

"If over 85 feet in height and not over 100 feet in height, the walls shall not be less than 28 inches thick to the height of 25 feet or to the nearest tier of beams to that height; thence not less than 24 inches thick to the height of 50 feet or to the nearest tier of beams to that height; thence not less than 20 inches thick to the height of 75 feet or to the nearest tier of beams to that height; and thence not less than 16 inches thick to the top.

"If over 100 feet in height each additional 25 feet in height or part thereof, next above the curb, shall be increased 4

inches in thickness, the upper 100 feet of wall remaining the same as specified for a wall of that height.

Or, by the ordinary method:

1st story	36 inches	7th story	24 inches
2d "	" "	8th "	" "
3d "	32 "	9th "	20 "
4th "	" "	10th "	" "
5th "	28 "	11th "	16 "
6th "	" "	12th "	" "

By the skeleton construction:

1st story	20 inches	7th story	16 inches
2d "	" "	8th "	" "
3d "	" "	9th "	12 "
4th "	" "	10th "	" "
5th "	16 "	11th "	" "
6th "	" "	12th "	" "

The height from floor to floor is generally 12 feet 6 inches.

Curtain-wall Supports.—The simplest supports for curtain walls are these girders made up of beams and channels, as shown in Figs. 82 and 84. These girders extend between the wall columns at about the floor levels and are made to receive the floor arch next the wall, as shown in the detail, which also shows the section of the sleepers, floor arches, and concrete filling.

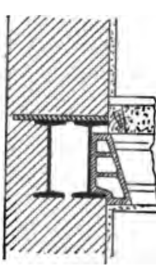


FIG. 82.

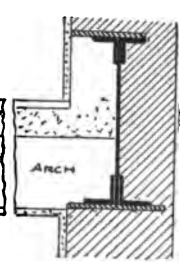


FIG. 83.

The outer beam of the girder is placed 4 inches from the party line, so that a width of brick can be built in to properly fire-proof the girder. Then, to support the overhanging portion of

the brick wall, a plate rests upon and is secured to the top of the girder.

Probably a better manner of building this outer 4 or 8 inch wall would be by that shown at the section of the channels, Fig. 84. The channel flanges are turned inside, so as to give a perfectly square and smooth surface for building this overhanging portion of the wall. Then again if the curtain wall occurs along the wall of a higher building the space back of the channel is clear, the plate being secured after the section is filled in.

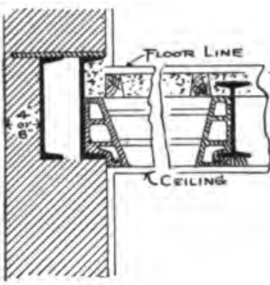


FIG. 84.

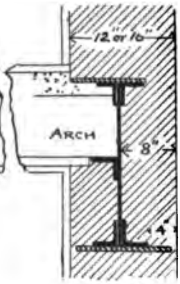


FIG. 85

This same thing may be accomplished in heavier walls when two beams are not sufficient and plate or lattice girders are used, as shown in the sections, Fig. 83. In Fig. 85 the plate girder is raised sufficiently to receive the floor arch upon the bottom flange; then the joint of columns is about the centre of the girder. In Fig. 85 an angle is riveted to the web of the plate girder to receive the floor arch.

The manner of connecting these wall girders is shown in the chapter on Column Connections.

The entire fronts of these skeleton buildings may be supported in such a manner that an entire story or number of stories may be removed without injury to the other portion of the structure—or, in other words, an exterior finish entirely independent of the rest of the construction, an example of which is shown by Fig. 86 (a section of the spandrel under the front windows of the Venetian Building, Chicago, Ill.); the outside view or perspective is shown by Fig. 78, Chapter VIII.

These spandrel beams are placed over the windows in such a way that all the load is taken off from the window-caps, however it may appear in the finish. The outside is covered with

brick, terra-cotta, tile, marble, or granite, or combination of these materials, as the architect may design, supported by the above spandrel beams. The section, Fig. 87, represents a portion of the front wall of the Ashland Block, Chicago, Ill.; the wall is only 8 in. thick, and the spandrel channel is 15 in. by 32 lbs. per foot, with an angle riveted to the upper edge to make a

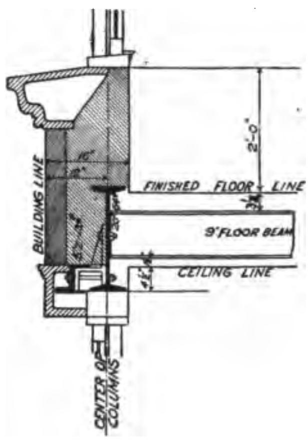


Fig. 86.

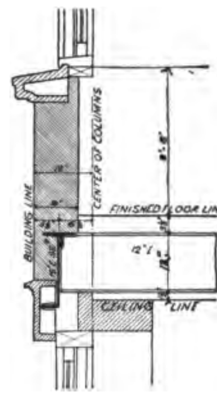


Fig. 87.

broad support for the wall. In Fig. 86 the wall is 16 in. thick, and to support the overhanging thickness cast brackets are secured to a 20 in. × 64 lbs. per foot I-beam, upon which is secured a 5-in. Z-bar. In each case the terra-cotta window head is secured to the construction.

Figs. 88 and 89 represent other modes of supporting the spandrel walls. Fig. 88 is another section of the Ashland Block, and Fig. 89 is a section of the spandrel walls of the Fair Building, Chicago. In all of these cases it will be noticed that the spandrel beams or girders connect to the columns near their centres; the building line represents the faces of piers or face of the building.

An excellent arrangement for radiators under the window-

sills is shown in Fig. 89; the spandrel wall is only 8½ in. in thickness at this point.

In almost all of the above sections the spandrel beams are so arranged, as to size and position, that the floor beams are

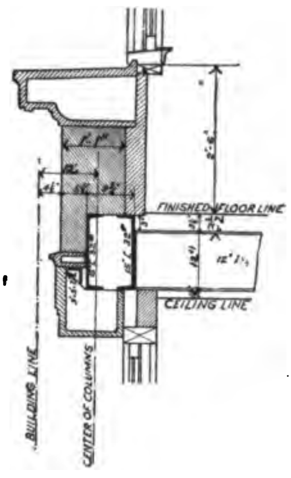


FIG. 88.

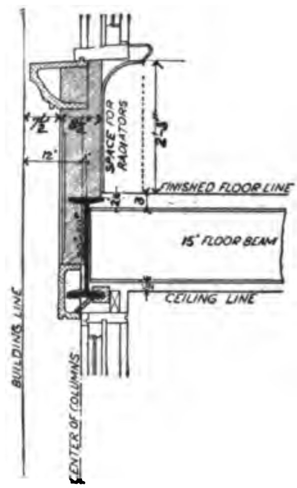


FIG. 89.

not required to be framed; this, if carried out extensively throughout the construction, will save considerable in the cost of the building.

CHAPTER X.

THE OLD COLONY BUILDING, CHICAGO, ILLINOIS.

THE Old Colony Building is situated on Van Buren and Dearborn streets, Chicago, Ill. It extends from Dearborn Street to Plymouth Place, thus having a frontage on broad streets of 368 feet. It is in the heart of the new business centre which has grown up on Dearborn Street, and is accessible to all suburban and street transportation, hotels, Post-office, Board of Trade, Custom House, and the United States Courts.

It was designed by Messrs. Holabird and Roche, architects, assisted in the construction by Corydon T. Purdy, C.E., to whom the author is indebted for the information. The exterior is of blue Bedford stone to the fifth story, and above of Philadelphia white brick with white terra-cotta trimmings.

The entrances from the three streets are finished in elegance and richness of design with marble and elaborate Italian mosaic. The interior finish is of the best, with the corridors all wainscoted in marble and the floors of mosaic.

The building consists of sixteen floors, basement and attic, of which the plan Fig. 91 is a typical floor. This plan describes clearly the arrangement of the offices, divided in a systematic and advantageous manner, and on account of the favorable site all the offices open to the outer air and are all connected to the wide and commodious corridor in the centre of the building.

Six large elevators with the newest appliances, constructed

of iron, extend from basement to attic and are arranged as shown on each side of the stairway. Directly to the right of

FIG. 90.—THE OLD COLONY BUILDING.

the stairs is the boiler-flue, which is constructed of iron and encased in brick.

In construction it is probably as perfect a type of the steel skeleton building as any that has been erected. There are no self-supporting walls, and all loads—brick, terra cotta, tile, and stone in walls and floors—are carried at each floor-level on the steel frame.

The special features of its construction pertain to the *cantilever* supports at the south end, the lateral strength of the structure, the column construction, and the protection against fire.

These have all attracted considerable attention from architects and engineers during the World's Fair months, and are deserving of special notice in this volume.

To protect the steel skeleton frame against fire, special precaution was taken, and all the columns were entirely surrounded with a 3-inch hollow tile wall, which in turn was covered, in the case of the outside wall columns, with a solid brick wall on three sides 13 inches thick. Then, again, these outside columns were placed 2 feet back from the street line, in contrast with columns which are usually placed 12 inches from the street line,—which is so common in that city,—and protected from any outside heat by only 4 or 5 inches of limestone or granite, or even of brick.

The Building Law of Chicago especially calls for the above mentioned protection:

"SEC. 101. Fireproofing of the steel and iron structural parts of buildings shall, for the purposes of this ordinance, be defined as follows: 'All iron and steel used for a supporting member of the external construction of any building exceeding 90 feet in height shall be protected, as against the effects of external changes of temperature and of fire, by a covering of brick, terra cotta, or fire-clay tile, completely enveloping said structural members of iron and steel. If of brick, it shall be not less than 8 inches thick. If of hollow tile, it shall be not less than 6 inches thick, and there shall be at least two sets of

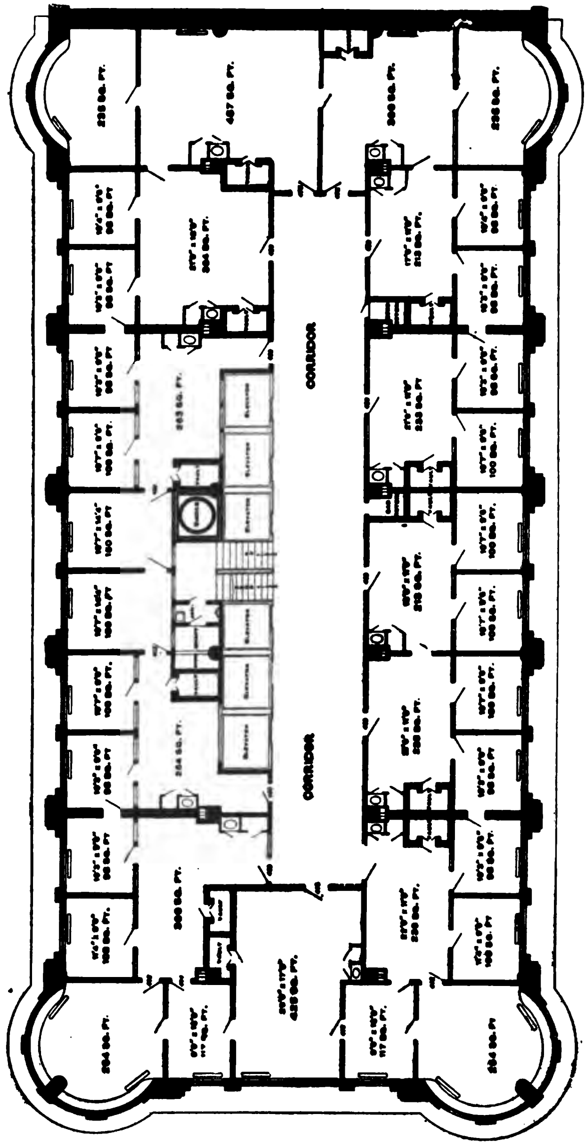


FIG. 91.—TYPICAL FLOOR-PLAN OF THE OLD COLONY BUILDING.

air-spaces between the iron and steel members and the outside of the hollow-tile covering. In all cases the brick or hollow tile shall be bedded in mortar close up to the iron or steel members, and all joints shall be made full and solid. Where *skeleton construction* is used for the whole or part of a building, these enveloping materials shall be independently supported on the skeleton frame for each individual story.'

"SEC. 102. If iron or steel plates are used in each story for the support of this covering within the said story, such plate must be of sufficient strength to carry, within the limits of fibre strain for iron and steel elsewhere specified in this ordinance, the enveloping material for the said story, and such plates may extend to within 2 inches of the exterior of said covering.

"SEC. 103. If terra cotta is used as a part of such fire-proof enclosure, it shall be backed up with brick or hollow tile; whichever is used being, however, of such dimensions and laid up in such manner that the backing will be built into the cavities of the terra cotta in such manner as to secure perfect bond between the terra-cotta facing and its backing.

"SEC. 104. If hollow tile alone is used for such enclosure, the thickness of the same shall be made in at least two courses, breaking joints with and bonded into each other.

"SEC. 105. The horizontal filling between the iron and steel vertical members of skeleton construction shall be of brick, terra cotta, or hollow tile, and, in case of less thickness than 12 inches, subject to the same conditions as to bond and courses as specified for the enveloping materials of structural members, and these horizontal fillings shall be bonded into the enclosures of the vertical members.

"SEC. 106. The upper surfaces of all breaks or offsets in external coverings and fillings and walls, as well as the tops of walls, shall be covered with stone, terra-cotta, or fire-clay

copings set in cement mortar and having lapped joints pointed with cement.

"SEC. 107. The internal structural parts of buildings of the skeleton construction shall be fireproofed by coverings of brick, hollow tile, porous terra cotta, or plastering on metal lath and metal furring.

"SEC. 108. In the case of buildings of Class I the coverings for columns shall be, if of brick, not less than 8 inches thick; if of hollow tile, these coverings shall be in two consecutive layers, each not less than 2½ inches thick. If the fire-proof covering is made of porous terra cotta, it shall consist of at least two layers not less than 2 inches thick each.

"Whether hollow tile or porous terra cotta is used, the two consecutive layers shall be so applied that neither the vertical nor the horizontal joints in the same shall be opposite each other, and each course shall be so anchored and bonded within itself as to form an independent and stable structure.

"In all cases there shall be on the outside of the tiles a covering of plastering with any cement—which is established as a standard cement by the society of civil engineers of the northwest—or of other mortar of equal hardness and efficiency when set.

"SEC. 109. In places where there is trucking or wheeling or other handling of packages of any kind, the lower five feet of the fireproofing of such pillars shall be encased in a protective covering either of sheet iron or oak plank, which covering shall be kept continually in good repair.

"SEC. 110. If plastering on metallic lath be used as fire-proofing for columns, it shall be in two layers. The metallic lath shall in each case be fastened to metallic furrings and the plastering upon the same shall be made with cement.

"Protection for the lower five feet shall be required in this case the same as where porous terra cotta or hollow-tile covering is used."

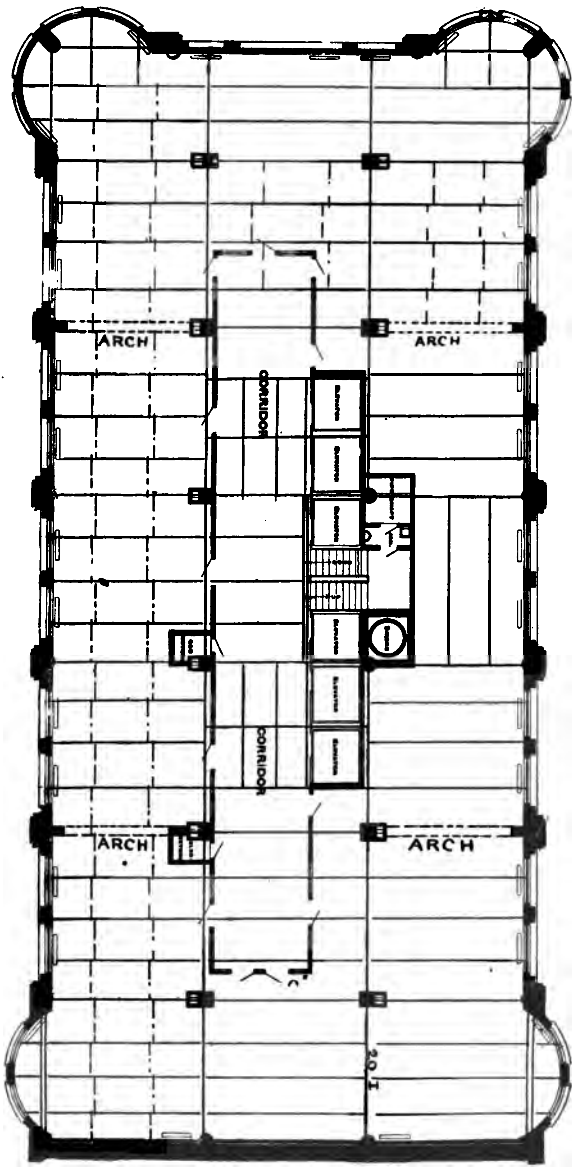


FIG. 92.—BEAM-PLAN OF THE OLD COLONY BUILDING.

THE LOADS USED IN CALCULATIONS FOR THE BUILDING, IN POUNDS PER SQUARE FOOT OF FLOOR.

	On Beams.	On Girders.	On Columns.	On Foundation Footings.
Live	70	50	40	
Dead	90	90	90	90
Total	160	140	130	90

The above 90 lbs. of dead load is made up as follows:

Floor-arches	35 lbs.
Concrete	18 "
Plastering	5 "
Flooring	4 "
Iron	10 "
Marble and Partitions	18 "
Total	90 lbs.

By referring to the plan Fig. 92 the arrangement of beams and girders is seen.

The outer lines of girders on the long sides of the building are formed of a 20-inch beam and angles. The two lines of inside girders running parallel to the above are 20-inch beams for the longer spans, 15-inch and 12-inch beams for the shorter spans. The columns supporting the girders are spaced about 22 feet apart. The floor-beams throughout the building are generally 12 inch by 32 lbs. per foot spaced about 5 ft. 6 in. apart; those adjoining the elevators for the short spans are 9 in. by 21 lb. and 6 in. by 13 lbs. per foot. The entire work being accurately and securely fitted with heavy knees.

When the planning of the building began, the support of the south end was the first really difficult problem encountered. The other three sides are bounded by streets, but this south end adjoins another property, which is occupied by an

old brick building six stories high with exterior walls and foundation built centrally upon party lines.

A new party-wall foundation extending somewhat over both lots, and large enough to carry its share of a new building on this neighboring property would, of course, have been the best and easiest solution of the problems. It could have been made without disturbing the old wall above the first floor, except to cut vertical openings in the outside wall for

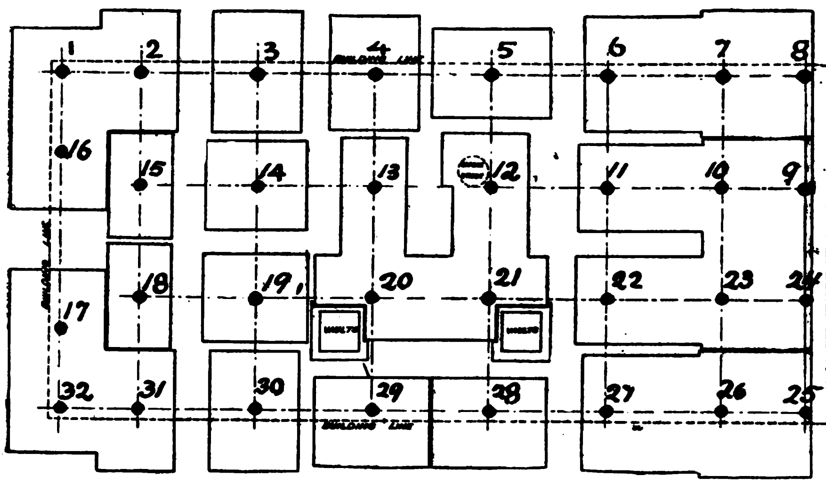


FIG. 93.—FOUNDATION-PLAN, SHOWING NUMBER AND POSITION OF COLUMNS.

the steel columns which an old party-wall contract permitted in any case. However no arrangement could be made to that end, and it became necessary to keep the foundations of the new building away from the old wall entirely or shorten the building. The cantilever construction was therefore adopted. The plan Fig. 93 shows the columns, their position and the clay areas of the foundations. There are thirty-two columns in all. All the foundations of the building are made of steel beams and Portland-cement concrete, several of them carrying three or more columns each. The areas are proportioned to

3200 lbs. per square foot of dead load, including the weight of the foundation.

This limit of loading made it necessary to include three columns in each cantilever foundation, and owing to the larger loads on the columns next the street, Nos. 8 and 25, it was necessary to combine the interior ones (9 and 24), making six columns in all on one area. Fig. 94 is a vertical section through the foundation for columns 25, 26, and 27. Column 25 is 3 ft. 6⅝ in. from the south party line and placed upon a triple-web box girder 2 ft. 6 in. wide by 2 ft. 6 in. deep, which in turn rests upon the ends of twenty-four beams 42 ft. 10½ in. long bedded in concrete. Under columns 26 and 27 there are eight 20-inch beams 22 ft. 4 in. and 20 ft. 3½ in. long respectively, upon which the cast-iron base of each respective column is bedded. This figure also clearly shows box-girder cantilever connecting with column 26, and the off-setting of the 25th column to its proper place for the supporting of the wall and floors above. This same construction applies to all the party-line columns.

All the other foundations throughout the building are arranged in the same manner as that shown in Fig. 114, page 236, of this volume, with the exception that very heavy beams were used and the height of the steppings limited to two layers of beams and the lower bed of concrete about 12 inches in thickness.

Chicago Building Law relating to setting of steel or iron beams in foundations:

"SEC. 79. If steel or iron rails or beams are used as parts of foundations, they must be thoroughly imbedded in concrete, the ingredients of which must be such that, after proper ramming, the interior of the mass will be free from cavities. The beams or rails must be entirely enveloped in concrete, and around the external surfaces of such concrete foundations there must be a coating of any cement mortar not less than one inch thick.

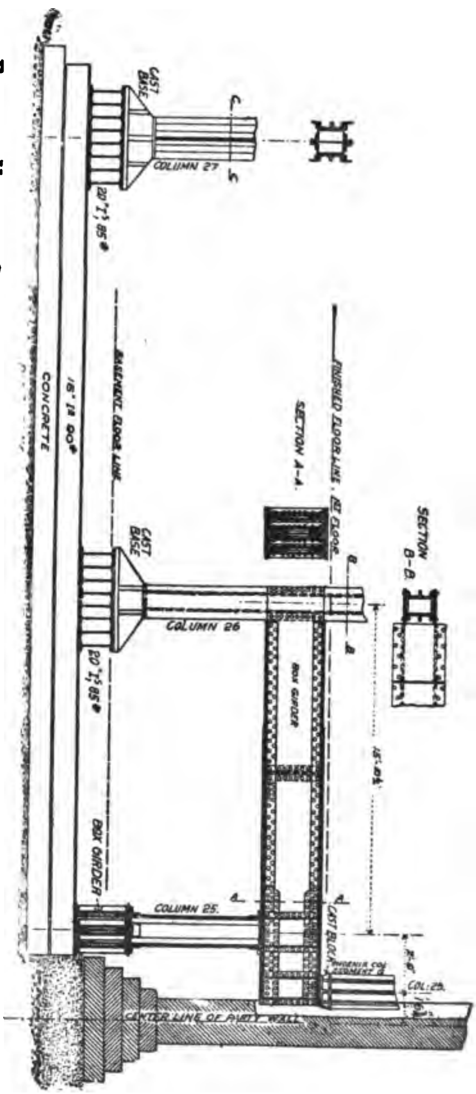


FIG. 94.—VERTICAL SECTION THROUGH FOUNDATION AT SOUTH PARTY LINE SHOWING CANTILEVER CONSTRUCTION.

"SEC. 80. If concrete foundations are used by themselves and without the insertion of beams, the offsets on top of same shall not be more than one half the height of the respective courses, and such concrete foundations must not be loaded more than 8000 pounds per square foot. If reinforced by iron or steel beams, the loads and offsets in the same must be so adjusted that the fibre strain upon the metal if of iron shall not exceed 12,000 pounds per square inch, or if steel, that the fibre strain shall not exceed 16,000 lbs. per square inch."

The calculations of the foundations for the three columns relating to each cantilever are much alike, and a description of the interior one may easily answer for all.

The clay load was determined by the following. The dead load includes floors, columns, and coverings, walls, etc.

	Live Load, lbs.	Dead Load, lbs.	Total on Basement Cols., lbs.
Col. No. 9	190,400	712,900	903,300
" 10	323,700	666,940	990,640
" 11	359,140	896,010	1,255,450
" 22	375,000	917,010	1,292,010
" 23	323,700	666,940	990,640
" 24	190,400	712,900	903,300

The additional load of the foundation itself is treated as though concentrated at column centres and in the same proportion as the loads carried by the column. This is not theoretically correct, though practically so, for the weight of the foundation, made as it is, is very evenly distributed, and therefore its centre of gravity should correspond with the centre of gravity of the loads. The actual weight of the foundations proved to be about 36,000 lbs. more than the estimated load, or about 20 lbs. per square foot more than the 3200 lbs. figured for.

Quite sufficient time was allowed for the preliminary study of this cantilever problem, but all the final calculations had to be made as rapidly as possible.

The column loads were obtained and various efforts were made to fix upon a footing that would bring the centre of gravity of loads and resistances together, but none could be made on a basis of 3200 lbs. per square foot on the clay. The plan shown fails of this by about $5\frac{1}{2}$ inches, the centre loading being that distance nearer the party wall than the centre of gravity of the clay area.

For various practical reasons, it seemed better to construct the work with this variation of centres than to recast all the foundations of the building to a basis which would not require such a variation, or do any of the several other things that might have been done.

After nine months, the average settlement of the foundations in the building is $4\frac{3}{16}$ inches, while the average settlement of the centre cantilever foundation is $5\frac{1}{8}$ inches, and columns 9 and 24 on the small side of the footing have settled an inch more than the average of the other four. The latter fact may be due in some measure to the $5\frac{1}{2}$ inches variation in load and resistance centres, although columns 9 and 24 had received 90% to 95% of their full load, while the other four columns had not received more than 75%, when these observations were taken. The explanation for the greater average settlement of the whole pile probably lies in the fact that this area is so completely and closely surrounded with the other foundations of the building and of the party wall of the adjoining building that most of the lines of resistance through the clay structure must necessarily be vertical, and all the advantage of its large perimeter is lost.

Wind-bracing—Portal Arches.—The lateral strength of this building has been provided for by four sets of *portal arches*

reaching from foundation to roof, as shown in the transverse section Fig. 95, being a section through columns 6, 11, 22, and 27. Two other sets similarly designed are placed between columns the same distance from the other end of the building.

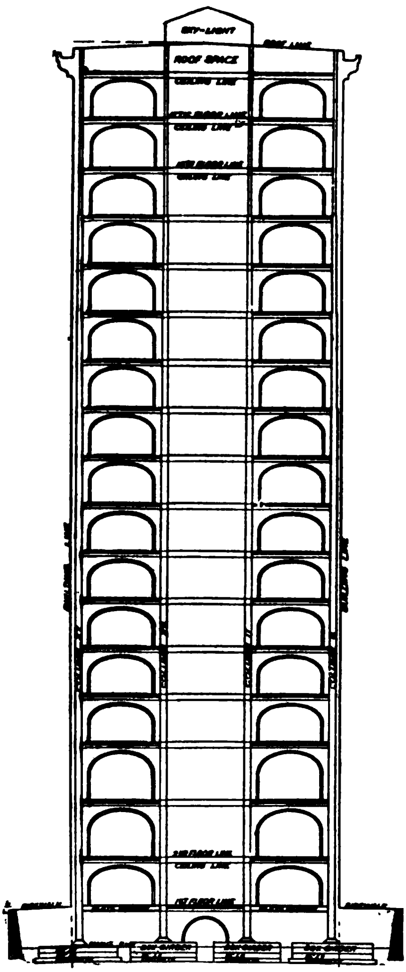


FIG. 95.—TRANSVERSE SECTION SHOWING PORTAL ARCHES.

The position which it was to occupy was fixed early in the work, but the portal construction was not decided upon until after the contract was let and it was determined to use Phœnix columns. The arrangement under which the contract was made was a combination of arches and tension-bars, and the change was made partially to save the eccentric load it brought on the columns, but largely for the advantage this system would have in the arrangement of the rooms, being unobtrusive and not injurious to renting interests.

The chief advantage of this construction is its adaptability. It may be put in almost any building somewhere without serious injury to the structure.

Even the stores and banking rooms of the building are arranged with the arches, so there will be no serious reduction of income nor an unpleasant appearance in the finish. The portal may be fireproofed so that the space on each side can be joined in one room, or if they are covered in a partition, doorways may be cut through to suit the arrangement of the rooms, except at the extreme sides. It is also believed that it will stiffen the building more than any system of tension rods against the minor vibrations to which Chicago buildings in particular are subjected. These vibrations are caused by street traffic or anything else that gives the neighboring ground a jar, and they are felt in Chicago more than elsewhere on account of the very mobile nature of the clay soil that underlies the entire business portion of the city.

In point of cost they will compare favorably with any system of *rod-bracing*, especially if the rods are designed so that doorways may be constructed through the partitions that cover them.

In this respect the rods are at a disadvantage in having to be doubled to resist wind from both ways, while all the metal in the portals is strained from whichever way the wind may blow. When these tension-rods are connected to the struts, as they generally are, the column strains are eccentric, while the portal construction, detailed as it is in this case, practically eliminates eccentricity of column strains from the top to the bottom of the system. The same result as the above in appearance could be obtained by using knee-bracing at the ceiling line as shown in Fig. 40, and constructing a light fire-proof arch partition, or suspending from the girder between the column light furring lath and covering with the plaster finish.

Sec. 123 of the Chicago Building Law calls for wind-braces in all buildings the height of which is more than one and one half times their least horizontal dimensions, and they should

be figured at not less than 30 lbs. for each square foot of exposed surface. The precautions against the effects of wind-pressure may take the form of any one, or more, or all of the following factors of resistance to wind-pressure: first, dead weight of structure, especially in the lower parts; second, diagonal braces; third, rigidity of connections between vertical and horizontal members.

The accumulated shears and the resultant column strains due to wind-bracing in the Old Colony Building are as follows:

	Shear.	Column Strain.
Roof and ceiling......	7,860 lbs.	4,220 lbs.
Attic floor............	19,580 "	15,660 "
16th "	31,620 "	33,390 "
15th "	43,570 "	57,820 "
14th "	55,760 "	89,400 "
13th "	67,900 "	127,170 "
2d "	185,120 "	1,006,990 "

The strains in the second row of figures apply to the columns carrying the floors given in the same line in the first row of figures.

They increase the regular load on the column away from the wind and reduce the regular load on the column next the wind.

It matters little how the initial loads were obtained so long as they were properly proportioned, for their full application would practically reduce the working column load, that is, the full dead load and a small live load, to zero. More than this would, of course, lift the columns.

They are equivalent to a pressure of about 27 lbs. per square foot over the entire surface of one side of the building at one time.

The inertia of the building, the strength of the exterior walls,

and the stiffness of the connections, especially of beams to columns, and the strength of partitions are all supplementary to this bracing, and probably make laterally one of the strongest steel-constructed buildings in the country. Each

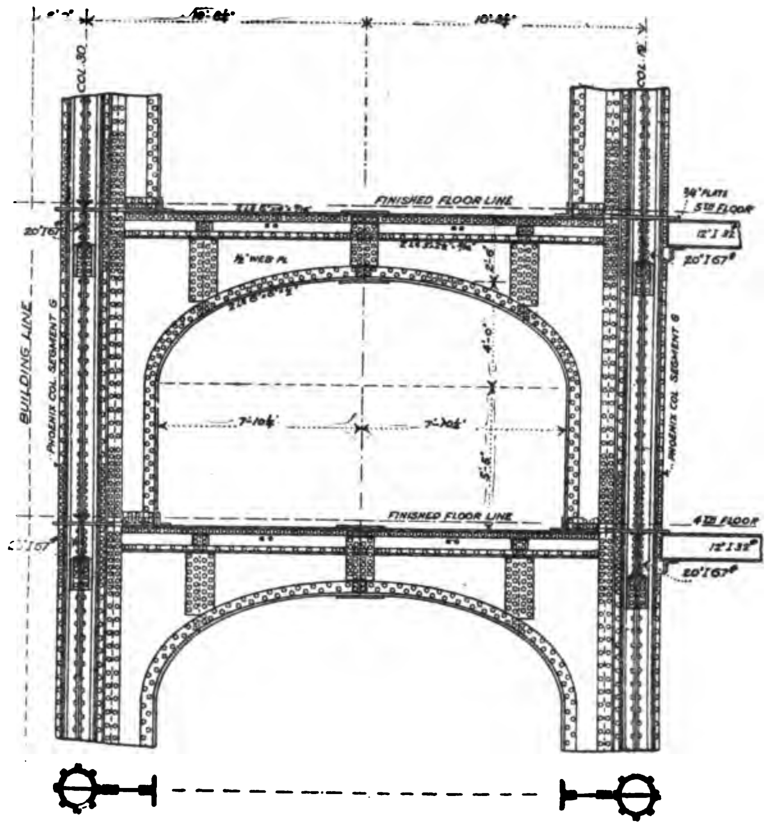


FIG. 96.—GENERAL ELEVATION OF PORTAL ARCHES.

portal was calculated independently of those above and below, for the sections of both top and bottom flanges, thickness of webs, cross-shear on rivets connecting curved flanges, and for splices and connections both to the columns and to adjoining

portals. The calculations were made as though the two halves of each portal were connected in the top or straight flange with an ordinary pin connection. In the actual construction, however, they were riveted together to secure simplicity in detail

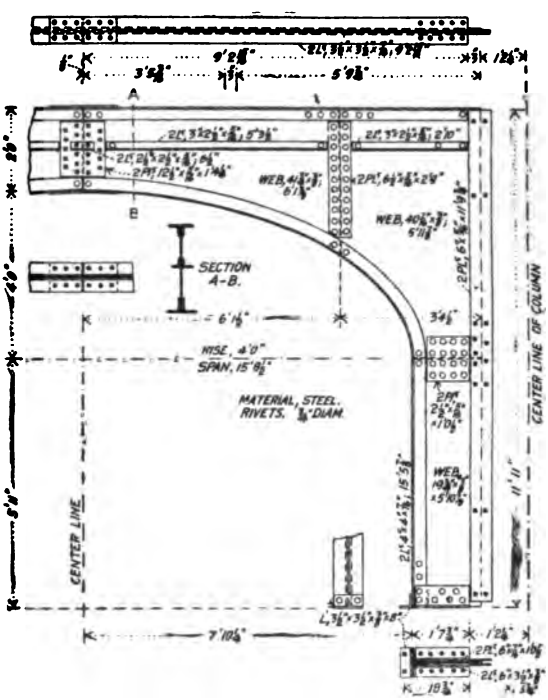


FIG. 97.—DETAIL OF ONE HALF ELEVATION OF PORTAL ARCH.

(see general elevation, Fig. 96), ease of erection, increased resistance to vibration, a stiffer member—which counts for a good deal during erection—because it carries a floor and to that extent must be treated as a beam. In Fig. 96 the sizes of arch and other measurements are given. The stories are about 12 feet from floor to floor, the outer columns are 2 feet from the building line, and the arches 15 feet 8½ inches be-

tween columns and 9 feet 5 inches high from top of horizontal member to crown.

Each portal was designed in two pieces as shown in Fig. 97, which is a shop drawing of one piece as it was delivered for erection.

The long splice outside the column was made in two pieces to admit of its erection after the other iron around it was in place. This proved a fortunate precaution, for the delivery of the portals was extremely slow.

The splices in the web on each side are in the interests of economy. The straight $3 \times 2\frac{1}{2} \times \frac{5}{16}$-in. angles are flush with the bottom of the regular 12-in. floor-beams, and are so arranged that they can carry the tile floor-arch. Each leg is attached to the portal below with three lug-angles and a rivet-section equal to one half the initial shear. The top and bottom plates in the centre have only a few rivets, but enough to make the member good as a beam, supporting a small floor-area, and to ease the erection somewhat.

The lightest metal used in the arches near the top of the building where the shears are so small is $\frac{5}{16}$ in. thick, the flange-angles being $3'' \times 3'' \times \frac{5}{16}''$. The design calls for a great quantity of field-rivets, but from what we understand it was such easy work that this riveting was not so costly as might be expected. The design is entirely new with this building and the "Monadnock," it being put into both buildings at the same time.

Referring to Fig. 98, the rest of the beam connections and column connections are shown. The specification relating to the detailing of the columns under which the contract was let is as follows:

"Beams connecting to columns shall have four rivets in the bottom flange wherever the details of the columns will permit of that number, and in all cases the beams must extend as

closely as possible to the axis of the column. The top connection angles connecting the beams to the columns shall be omitted on all floors, and bent strips of heavy sheet-iron or especially designed wedges must be driven in at the end of

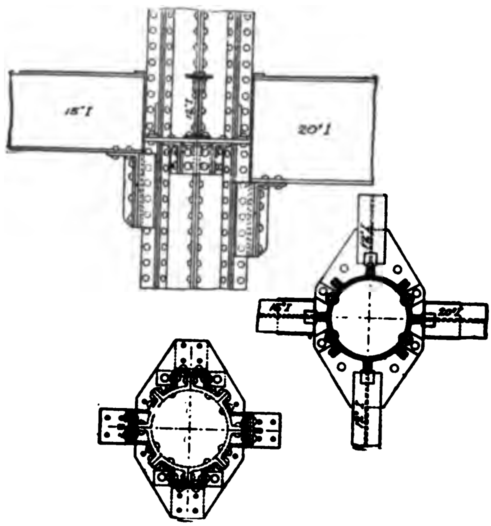


FIG. 98.—DETAIL OF COLUMN CONNECTIONS.

each beam in place of the top connection angle until the clearance between the end of beams and the metal of the columns is tightly closed. In case the flange of the columns will not serve to hold these wedges in place, some other means must be employed to serve the same purpose.

"All columns shall be provided with cap-plate $\frac{3}{4}$ inch thick, and, in general, the columns shall be cut so the floor-beams of girders shall rest directly upon them; dependence, however, must not be placed entirely on the shear of the plate to carry the beams. . . . Columns shall be connected to columns, when possible, by at least four rivets passing through the cap-plate, and two lug-angles, one on each column. All rivets used in

connecting beams to columns must pass through the lug-angles connected directly to the columns, and riveting such connections to the cap-plate without a lug-angle will not be allowed."

The ordinary Phœnix columns did not conform to the specifications, and the only system of connection in use to any extent which it was hoped could be made to apply was the Phœnix improved column, of gusset-plates and fillers, as described page 57, Fig. 37. There were, however, immediately three objections: first, the Phœnix Company objected on the score of cost, it being too great for the price they were to receive per pound; second, it would have added materially to the tonnage; and third, the great irregularity of beams, not only in the spandrels but in the floors both as to height and as to position relative to the axes of the columns, made even that system seem impracticable. Finally a general scheme was devised for these connections as mentioned before (detail Fig. 98).

In a few cases where heavy spandrel loads had to be carried 16 or 18 inches from the centre of the column, gusset-plates and fillers were introduced. The gusset-plate without the fillers was also used in all wind-bracing columns to connect to the portals. The gusset-plates in all cases extended the entire length of the column.

The architects have broken up each long façade of the building by inserting at each end a circular bay (see plans and perspective). The metal construction of one of these bays is shown in plan view Fig. 99, and a small section also shown on the same figure. Twelve-inch heavy beams connect with each column; to these beams, cantilevers composed of plates and angles are secured as shown, and constructed in such a manner as to keep the floor and ceilings level.

The column in bay has a bracket with a gusset-plate extending through the column. To the outer ends of the

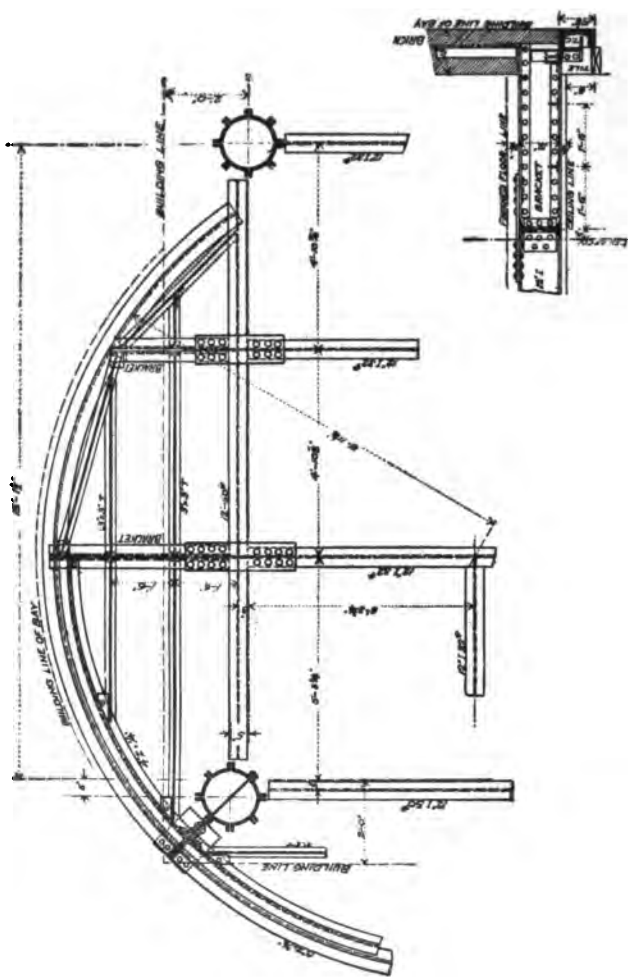


FIG. 99.—CONSTRUCTION OF CORNER BAYS.

bracket bent channels and Z bars are secured which support the sprandrel wall between each story.

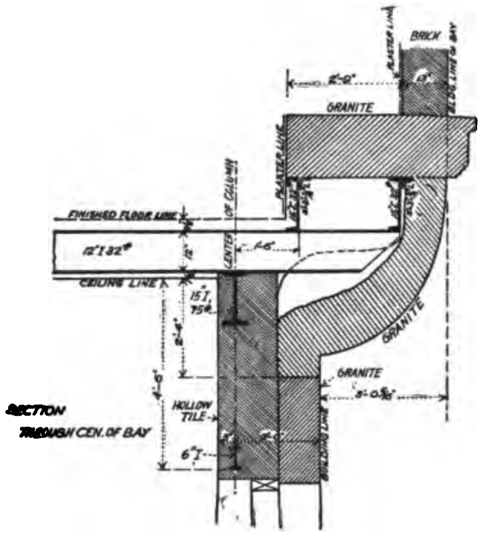


FIG. 100.—SECTION OF BAY.

Fig. 100 shows a section through the centre of the bay at the base. A seat of solid granite extends around on top of bent 15 inch channels.

CHAPTER XI.

THE MANHATTAN LIFE INSURANCE BUILDING, N. Y.

THE new building erected by the Manhattan Life Insurance Company at 64, 66, and 68 Broadway, New York, is undoubtedly one of the most conspicuous and the highest office-building in the world. On a comparatively small plot of ground 67 feet front on Broadway, 119 feet deep on the north line to New Street, and 125 feet on the south line, Kimball & Thompson, architects, and C. O. Brown, civil engineer, of New York, have designed and constructed a building of the skeleton type 16 stories high on the Broadway front and 17 stories on New Street. It has a height of 242 feet from the Broadway sidewalks to the top of the main roof and a height of 254 feet 4 inches on New Street. From the main roof on the Broadway front rises a tower terminating in a dome, which increases the height of the building from the Broadway curbstone to the foot of the flagstaff to 348 feet. The style of the Broadway and New Street fronts is Italian Renaissance richly ornamented. The Broadway front is of limestone, and the New Street front of light-colored brick and terra cotta; the side walls are of brick and supported as is usual in the skeleton frame.

The special feature of the Broadway front is the arched doorway extending through two stories, with a recessed vestibule of stone extending back in the building 13 feet, the sides and ceiling being richly ornamented. The spandrels of the arch have cartouches, on which are inscribed the

FIG. 101.—THE MANHATTAN LIFE INSURANCE BUILDING, NEW YORK.

date of the foundation of the company, erection of the building, together with the seal of the company. The other special features are the sixth and seventh stories, which are designed to emphasize the location of the offices of the company, and which are especially marked by the recessed arcade and the projecting balcony.

In the design the architects have aimed to preserve as much as possible a solid and dignified character and to avoid excessively large openings. From the sixth story upward the front is more irregular and is marked by side pavilions, the central portion being slightly raised. These pavilions terminate in small domes above the main roof.

At the level of the fourteenth story the front recedes from the front line of the building for the width of the central portion and is carried back to the face of the tower, which stands feet 7½ in the rear of the front. The inside of offices are lighted from a large open court on the south side of the building, thus giving every office abundant light and air.

On the sixth floor there is a spacious rotunda two stories in height, with a domed ceiling richly decorated in relief. This rotunda is designed for the public entrance to the company's offices. There are five hydraulic elevators for the use of the public and two electric elevators for the use of the company.

Careful attention has been paid throughout to the fire-proof qualities of the building.

There is no metal work exposed to the action of fire, all being covered with fire-proof materials.

All the staircases above the first story to the eighth floor are of marble and iron; above that of slate and iron. The elevator fronts are of cast iron and wrought grille-work heavily electroplated. All the floors, halls, and corridors are laid with mosaic. Marble, concrete, hollow brick, and

tile are largely used throughout the other portions of the building.

A ventilating chamber is formed overhead in each corridor (but not in elevator halls), by suspending from the floor-beams above two angles, one on each side, running the entire length of corridors and to the ventilating shaft, with which the chamber connects. On these angles are placed $3'' \times 3''$ tees, set 20 inches on centres, for holding porous terra-cotta blocks. Each office is connected with the above chamber by registers under the control of the tenant. At the head of each ventilating shaft there are electric exhaust fans, supplying the motive power for the extraction and discharge of the vitiated air.

The heating and power system is supplied by three marine boilers placed under the sidewalk on Broadway.

The care taken in the manufacturing and designing of the steel skeleton frame bore fruit in the erection of the work.

The first material was set September 1, 1893, and the setting of the roof-tier was completed December 1, 1893. In spite of the rapidity with which the work was prosecuted, the only accident recorded against it was the dropping of a small girder from the roof, which caused but little damage.

The total weight of the iron and steel work in the building amounts to 5800 tons.

Some of the sections are of an enormous size and weight. The cantilever girders in the basement are 65 feet $10\frac{7}{8}$ inches long, 3 feet 4 inches wide, and 8 feet deep. They weigh eighty tons each. They came to the building in four sections 10 inches wide and the same length as above. The cantilever girder over the second story on New Street is 66 feet long, 2 feet 6 inches wide, and 4 feet 6 inches deep. It weighs forty tons.

The front of the building is self-sustaining, that is, it is calculated to support its own weight, but not that of the

floors. The pressure at its base is such that it was necessary to carry the base up to the Broadway level of solid granite. All the other walls of the building, including the New Street front, are carried at the floor levels upon steel girders inserted between the columns.

Office Arrangement.—The subdivision of the rentable space of a building into offices determines at once its financial success. Therefore a good office building should contain, as belonging to the above requirement, good light, ease of access, good service, etc.

New York real-estate men state that offices containing 150 to 250 square feet are always to be rented in a desirable building, and the large majority of office buildings are so divided as to permit of the renting of such small units.

That being the case, those offices containing over the above amount must inevitably be more difficult to rent.

In referring to the plan Fig. 102, it will be seen at once that such requirements as the above have been well applied in this building. The rooms are convenient, well lighted, and open into agreeable halls which are also well lighted and accessible.

Every possible necessity has been provided.

The main elevators, five in number, as well as the stairway, are placed in a most desirable position. The division of the offices has been based on the experience of the latest and best examples of commercial buildings, and the interior court, which is believed to be the largest for the size of the building of any in existence.

Arrangement of Beams and Girders.—The building is calculated to sustain upon every superficial foot of floor and roof surface 175 pounds—the standard fixed by the present Building Law of New York. This includes the weight of beams, floor-arches, hollow-block partitions, furniture, ordinary safes, and the weight of people occupying the building, sufficient

allowance being made to have it crowded with people as closely as they can be packed.

In addition to the above, special provisions have been made to sustain the concentrated weights of the large vaults

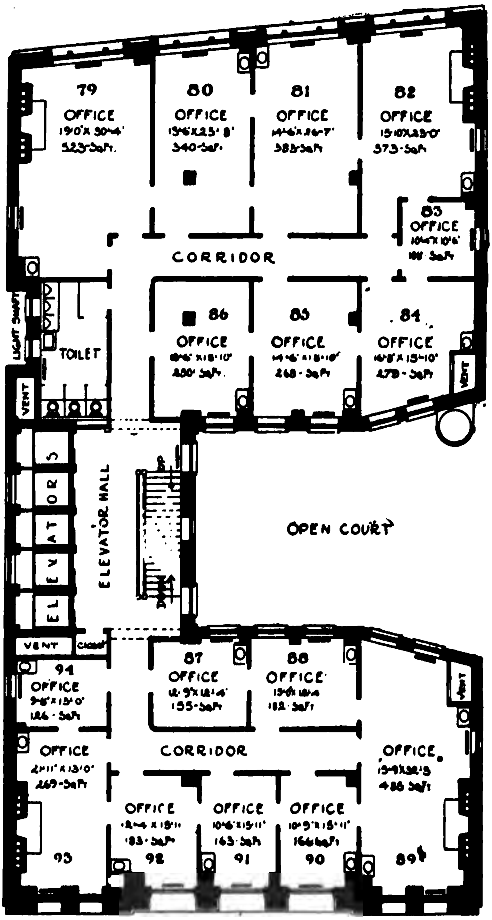


FIG. 102.—TYPICAL FLOOR-PLAN.

which are located in the basement, and also the vault for the company's use at the fifth floor.

In both practical and theoretical solutions the best of the

Chicago buildings seem at once by comparison more successful than those of New York, in that the former deal with actual

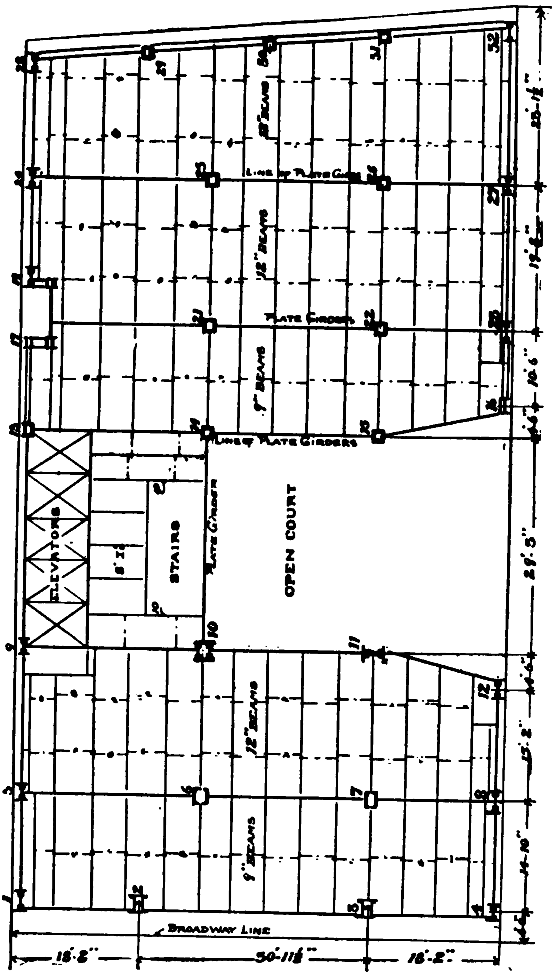


FIG. 103.—TYPICAL BEAM AND GIRDER PLAN.

loads and actual conditions, and the steel and masonry work is exactly proportioned to the duties to be performed; whereas in the latter, on account of the provisions of the building law,

the buildings are more massive structures. The difference in principle between Chicago and New York practice is not only confined to floor and column loads, but also to the thickness of walls, which involves heavier foundations and heavier columns, beams, girders, and consequently *larger bills to pay.*

The beams in this building are proportioned to carry nearly the same load as that required by the Chicago Law. The longer spans have 12-inch and the smaller spans 9-inch beams placed about 4 to 6½ inches apart.

The different classes of girders used throughout the building are known as single-plate, double-plate, box and lattice-truss girders. The single-plate girders 20 inches deep are generally used for the support of the floor-beams. Those for the support of 12- and 16-inch walls and the beams resting thereon are generally 24 inches deep.

These wall-girders, as shown in the detail section Fig. 104, are supplied with stiffeners at the ends and at intervals in the length of girder of not over three feet between centres.

The double-plate and box girders are used for the support of 20-inch walls and over. Those for the 20-inch walls are spread to make 15½ inches in width over flanges; those for 24-inch walls to 18 inches over flanges; those for 28-inch walls to be 20 inches over flanges; and for greater thickness of walls are made in like proportion. The stiffeners of the 20-inch and 24-inch wall-girders are shown in the detail, and spaced the same as in single-plate girders. All double-plate girders are supplied with *stay-plates* on top and bottom sides, $9'' \times \frac{5}{8}''$, of lengths sufficient to cover over flanges, and are riveted to each flange-angle with three rivets. These plates are staggered in spacing, so that the upper plate comes over the centre of space between the lower ones, which allows the brick walls to be built through the girders and retain a proper bond.

The row of girders in New Street wall (see cross-line section) at level of sixth floor, the flanges of which are too close to

admit of the width of a brick, have been run in solid with concrete.

The box girders are of two kinds—those made of two webs, angles and cover-plates, and those of a series of single-web

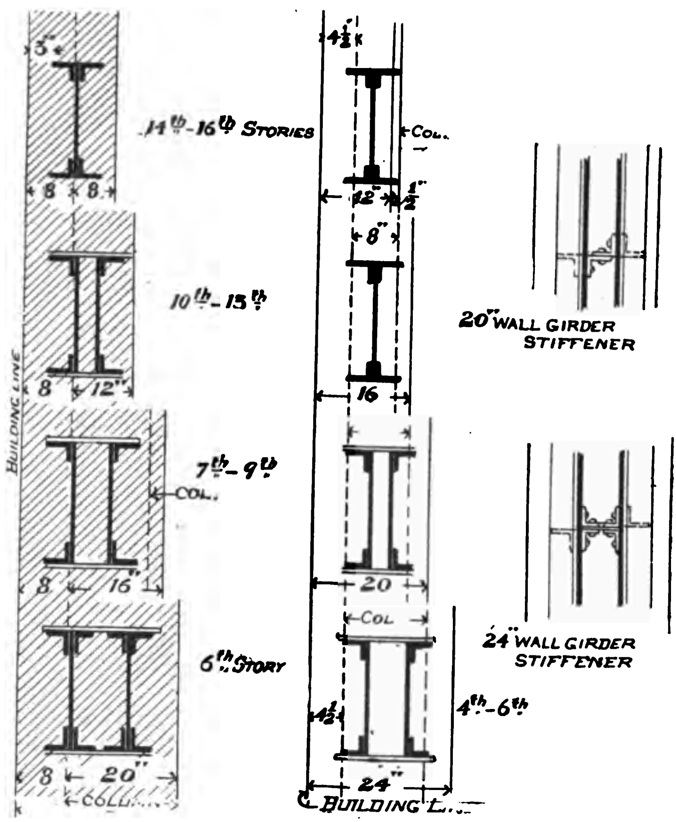


FIG. 104.—SECTION OF NEW STREET AND SIDE WALLS.

girders, bolted together with separators between, and placed at intervals of not over three feet on centres.

Cast-iron Columns.—Throughout the lower stories and largely through the interior of the building cast-iron columns have been used. The requirements of construction were that

they should be castings of uniform quality and subject to the following test: sample pieces one inch square and five feet long cast from the same heat in sand moulds, placed on supports 4 feet 6 inches apart and capable of sustaining a centre load of 500 lbs. when tested in the rough bar.

The flanges of columns were turned true in the lathe to

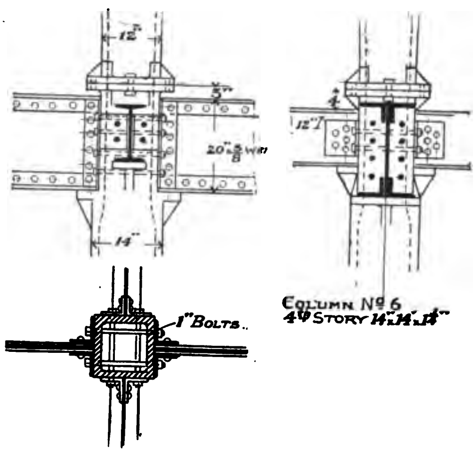


FIG. 105—CAST-IRON COLUMN-JOINT DETAIL.

exact lengths as required, to make a perfect contact, and the thickness of flanges was the specified thickness after planing.

The detail Fig. 105 shows the system of connections adopted for the cast-iron columns generally throughout the building, The seats for beams and girders are cast with the column, project 5 inches, and are 2 inches thick.

All girders and beams when resting on the above seats are connected together by one-inch bolts running through the column, the holes for which were bored through steel templates, the templates then used for reaming out corresponding holes in end-stiffeners of girders and in angle-iron lugs of beams.

When the sectional area of cast-iron columns called for interior webs, said webs were cast the same thickness of metal

as the outer shell, and the area of section maintained throughout the full length of column.

Where a smaller column rests upon a column of larger size the *core* of the larger column at the place of contact is the same size as the core of the smaller, and the metal tapered down for a distance of at least six inches.

The bolt-holes in flanges were accurately cored $1\frac{1}{8}$ inches in diameter for one-inch bolts.

The cores for the columns were rounded off at the corners to a one-inch radius; the outer corners were rounded off to a radius of $\frac{1}{4}$ inch.

Ample fillets are provided at all corners of flanges, lugs, and brackets, excepting where interfering with connections of beams and girders.

Steel Columns.—The steel columns throughout the building, as shown on the beam plan, are composed of Z bars, angles, channels, and such combinations of shapes as indicated. The different members are all riveted together by machine and made in lengths about 40 feet long, or equal in most cases to three stories in the height of building.

All abutting ends are planed, the joints fully spliced with steel plates and cover-angles, and when the columns were placed in position were securely riveted together. All seats for beams and girders consist of $6'' \times 6'' \times \frac{1}{2}''$ angles, as shown in the detail Fig. 106, and riveted to the column; a similar angle of corresponding size is provided on top side of girder as shown; it is also riveted to the column, beam, and girders.

Where a steel column starts upon one of cast-iron, the foot of the steel column is reinforced by plates and angles and riveted to a wrought-steel plate, which latter is bolted to the flange of the cast-iron column with one-inch bolts. The splices are arranged so that they come above and as near the floor-level as practicable; the full section of the lower heavy column

extends up to the joint. The splicing of one of the lighter side-wall columns is shown in detail, Fig. 106.

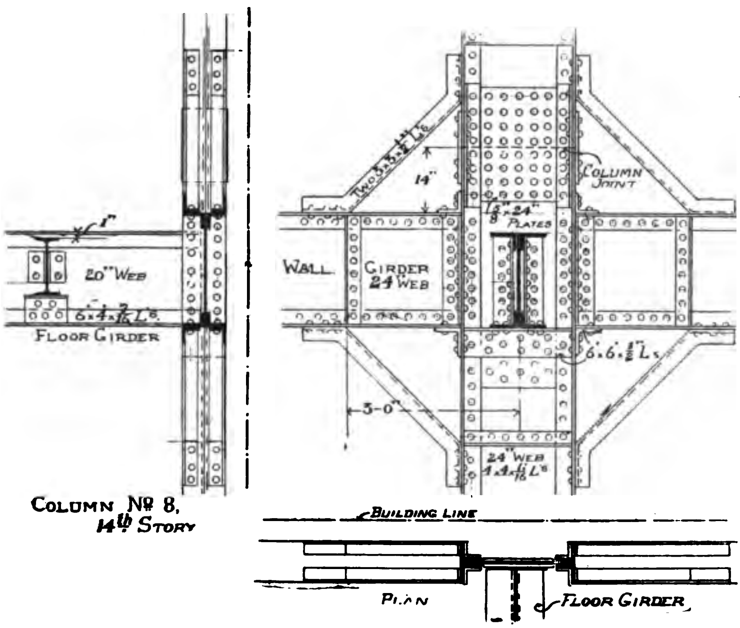


FIG. 106.—STEEL COLUMN-JOINT DETAIL.

In proportioning the splices of the other columns, the area of the splices, plates, and angle-splices is equal to the area of the next column above, and the number and size of rivets are such that they transmit the full strain of the upper column. Where the section of the lighter columns is thinner the difference is made up by filling-plates of proper size and thickness.

Where the steel columns are made up of plates and channels, they are put together in box form, as nearly square as practicable, and the channels placed with flanges facing each other and connected together with lattice-bars $2\frac{1}{2}'' \times \frac{3}{8}''$ of flat rolled steel. The bars are set at an angle of 60° to the axis

of the column and riveted to the flanges of channels. The latticing on one side is run in a direction opposite to that on the other.

Where the columns are made of more than one web with no cover-plates, the web-plates are spread as indicated on wall-section, Fig. 104, with stay-plates 9″ wide by the width of the column, and of a thickness equal to the angle-iron used in the column, placed not over three feet from centres on each side, and riveted to each flange with three rivets.

In addition to stay-plates there are angle-stiffeners placed every three feet apart each side of the web, ½ inch less than the general size of angles used in the column, and in thickness ⅛ less, but none less than $\frac{5}{16}$ inch. Filler-plates are placed back of each angle-stiffener which completely fill the intervening space to web.

The detail shows in plan view angle knee-braces of 3″ × 3″ × ½″ L's and position of wall line, which is 4½ inches inside the building line. It also shows the 20-inch floor-girder and beam connection to the same.

The girders are riveted to the columns as shown in the detail, and the number of rivets forming this connection is such that the shearing strain on the rivets does not exceed 9000 pounds per square inch.

The rivets in the girder-stiffener are calculated to carry the entire load.

Riveting.—The rivets throughout the building are generally ¾″, ⅞″, and 1″ in diameter. The size used, however, is not less than the thickness of the heaviest member through which the rivet passes. The rivet-pitch is not less than three times its diameter, nor more than 6 inches, and proportioned to sustain the loads on columns, girders, and beams without being strained to exceed 9000 lbs. per square inch in shear or 15,000 lbs. per square inch on the bearing surface. No rivets are closer to the edge of any member than 1½ inches for one-

inch rivets, $1\frac{5}{8}$ for seven-eighths rivets, and $1\frac{1}{4}$ for three-quarter rivets.

Cast-iron Lintels.—Cast-iron lintels were provided over each and every opening in outside walls and where walls were of masonry. The width of lintels over the windows of side walls is 4″ less than the thickness of wall. In the court they cover the entire wall and sustain the brick head. The outside face is neatly moulded. Those for New Street are nearly the full thickness of wall and support the terra-cotta head. Those for Broadway are governed by the reveal of the granite, and support the full brick wall back of granite. The thickness of metal up to a four-foot span is $\frac{3}{4}$ inch, up to five-foot spans, $\frac{7}{8}$ inch, and above five feet 1 inch.

Framing in Fire-proof Block Partitions.—All openings of doors and windows in block partitions are framed of 4-inch channel uprights extending through the height of story, and secured top and bottom by 3-inch angles to beams or girders as the case may be. A similar channel is placed horizontally at the heads of all doors, at the heads and sills of all windows, with the flanges turned outward in all cases, to hold the blocks of partition in position.

Anchoring of Walls. — Spear-anchors were provided throughout the building above the level of adjoining buildings, of $1'' \times 1\frac{1}{2}''$ flat iron with $\frac{5}{8}$-inch spear ends, to tie all walls passing in front of the wall columns on the outside, and were placed every 5 feet in height on each side of the columns. Similar anchors were provided at the top and bottom flanges of wall-girders at intervals of 5 feet, and so placed in the walls that the spears were vertical.

Heavy anchors were provided in the Broadway front, formed of $4'' \times 4'' \times \frac{1}{2}''$ steel angle double lugs riveted to the columns every 5 feet vertically, with $3'' \times \frac{1}{2}''$ flat steel bars bolted between each pair of lugs and extending out as far as the granite facing permitted, with a spear 1 inch in diameter

and 24 inches long. The New Street wall and court walls were provided with the same anchors as used in the side walls.

Arcade at Fifteenth and Sixteenth Stories.—At the fifteenth and sixteenth floor-levels there is constructed an arcade, as shown upon the perspective Fig. 101, a skeleton structure supported upon two latticed arches of heavy 6″ × 6″ angles and latticing connected at the ends by bolts running through columns 12 and 16. These arches are connected together on top with 5-inch steel beams, and at bottom by 3″ × 3″ angles spaced 3 feet apart. The entire framework is covered with copper, and not only improves the appearance of the south side, but acts in a great measure as a suitable brace for the upper portion of the building.

Tower and Dome.—The recessed portion of the fifteenth and sixteenth stories of the front is built upon four plate girders running parallel with the front and supported upon two trusses, shown at Fig. 107. The structural work of the

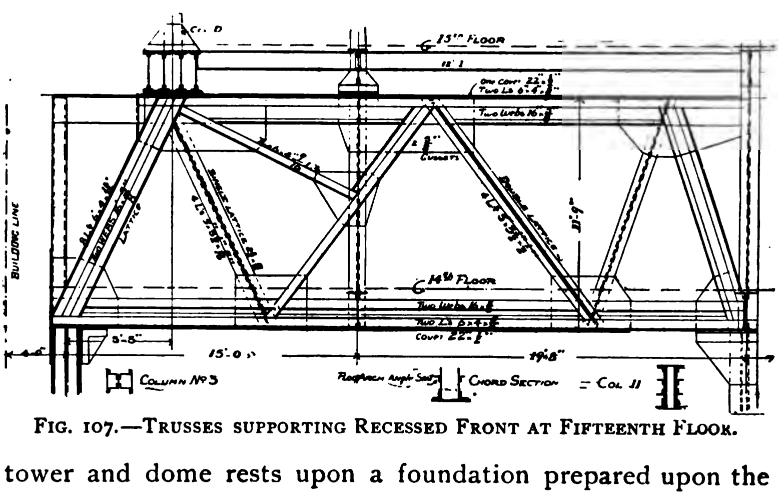


FIG. 107.—TRUSSES SUPPORTING RECESSED FRONT AT FIFTEENTH FLOOR.

tower and dome rests upon a foundation prepared upon the level of the main roof and upon the columns which are supported by the above trusses.

From this foundation twelve columns start, made of plates

and angles and latticed with steel angles. These latticed angles, and diagonal angles to resist the wind-strains, are placed between the columns and also on the outside, and connected to the columns and to each other by connecting plates at the intersections.

These columns form the tower, 29 feet 9½ inches square by 34 feet high, up to the beginning of the circular portion. At this height they are connected to a box girder extending around the four sides of tower.

The floor at the beginning of the circular portion is framed with plate girders and beams, upon which the eight latticed ribs rest. This circular frame is 24 feet in diameter by 27 feet high; the ribs are constructed of four angles $4'' \times 4'' \times \frac{3}{8}''$ with single latticing on four sides of $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ L's. Eight arched ribs start from the eight ribs before described, 26 feet 9½ inches high, terminate and butt against a circular girder made of a steel plate $18'' \times \frac{1}{2}''$, and two angles $6'' \times 6'' \times \frac{1}{2}''$ fully spliced and bent to a circle.

The eight arched ribs are made of four flange-angles $4'' \times 3'' \times \frac{3}{8}''$, double-latticed with angle-bars, $3'' \times 2'' \times \frac{1}{4}''$, all securely riveted together. The head and foot of arched ribs are reinforced by plates and angles. From the foot of the arched ribs an eight-inch steel pipe starts, and extends through the lantern.

The lantern is 5 ft. 9 in. in diameter, 14 ft. high, and formed of eight $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles riveted together in pairs, bent to a circle at the top and connected to the compression-ring by steel plates.

The dome portion of the tower is covered with $3'' \times 3'' \times \frac{3}{8}''$ tees, bent and twisted to the shape of the dome, placed 20 inches between centres to hold terra-cotta blocks.

The entire exterior facing of the tower and dome is covered with cold-rolled copper upon fire-proof block. All cornices, mouldings, and ribs are secured to wrought-iron brackets.

The entire height of tower, dome, and lantern, from the roof-level, is 101 feet 9½ inches, and is constructed with ample rigidity to resist a wind-pressure of 50 pounds per square foot upon its surface blowing in any direction. The rest of the building is calculated to withstand a pressure of 30 pounds per square foot.

Foundations by the Pneumatic Process.—The great height, the massive metal and masonry construction, impose enormous loads on the foundations, amounting to as much as 2000 tons for some single columns, and giving about 7300 pounds per square foot on the whole area of the lot. For this reason the so-called pneumatic process of sinking a pier was adopted; and the cantilever principle, so well known in bridge construction, has been employed in distributing the load of the column proper over the piers formed by caissons. This is probably the first time this construction has ever been employed for carrying down the foundations of a large building, although common enough in the construction of bridge piers and foundations in or near the water.

The enormous weight referred to above could not be safely carried on the natural soil, upon which this site, which is essentially of mud and quicksand to the bed-rock. The latter has a fairly level surface about 54 feet below the Broadway street-level.

Above this rock the water percolates very freely, standing at a level of about 22 feet below the Broadway level.

If piles had been driven as close together as the city regulations permit—i.e., 30 inches centre to centre over the whole area,—about 1323 might have been placed, and would have carried an average load of 45,300 pounds each, which was inadmissible, the statute law of New York allowing only 40,000 pounds each on piles 2 ft. 6 in. apart and with a smallest diameter of 5 inches. Special foundations were therefore necessary, and it was imperative that their construction and duty

should not jeopardize nor disturb the existing adjacent buildings.

On the south side the six-story Consolidated Exchange Building is founded on piles, which are supposed to extend to the rock. On the north the foundations of a four-story brick building rest on earth about 28 feet above the rock, and were especially liable to injury from disturbances of the adjoining soil, which was so wet and soft as to be likely to flow if the

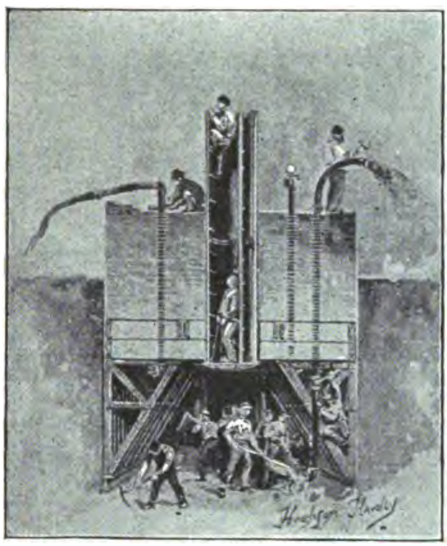


FIG. 108.—SECTION SHOWING MANNER OF EXCAVATING IN CAISSONS.

pressure was much increased by heavy loading or diminished by the excavation of pits and trenches. It was determined, therefore, to carry the foundations on solid masonry-piers down to bed-rock. The construction of the piers by the pneumatic-caisson process adopted was after careful consideration by the architects, backed by opinions from prominent bridge engineers as to its feasibility.

In executing the work an excavation about 28 feet below

grade (to water-line) was made over the whole area of the lot. Then the steel caissons were received, the smaller ones complete and the larger ones in sections, bolted together when necessary, and located in their exact horizontal positions, calked and roofed with heavy beams to form a platform, on which the brick masonry was started and built up for a few feet before the workmen entered the excavating-chamber and began digging out the soil. See the following vignette, which shows a vertical section through caisson, pier, air-lock, and shaft, reprneseting the excavators at work and shovelling mud into the foot of the blowpipe, from which it is ejected above. One man is stationed in the chamber at the valve to close it as soon as the air begins to escape. (See *Engineering Record*, Jan. 20.)

The removal of the soil allowed the caissons to gradually sink to the rock below, without disturbing the adjacent earth, which was kept from flowing in by maintaining an interior pneumatic pressure slightly in excess of the outside hydrostatic pressure due to the distance of the bottom of the caisson below the water-line.

The adjacent buildings were shored up at the outset and scrupulously watched, observations being made to determine any possible displacement or injury of their walls, which were not seriously damaged, though the pressure they exerted on the yielding soil tended to deflect the caissons which were sunk within a foot of the walls.

The caissons encountered boulders and other obstructions, and were sunk through the fine soil and mud at an average rate of four feet per day. No blasting was required until the bed-rock was reached and levelled off under the edges, and stepped into horizontal surfaces throughout the extent of the excavating-chamber. Usually one caisson was being sunk while another was being prepared, there being only one time when air-pressure was simultaneously maintained in two cais-

sons. Generally about eight days were required to sink each caisson.

Before beginning the caisson-work the adjacent wall of the building north of the lot was temporarily supported by the insertion of needle-beams to permit the removal of the old footing, which was replaced by a new concrete footing, about 10 feet wide by 4 feet high, which formed a continuous foundation for that wall, and also for the lower part of the light side-wall of the new building.

The caisson, considered as an aid in sinking foundation through wet material, consists of an inverted box having a sectional shape according to the work it is intended to do—sometimes circular—as shown under column 5, which is 13 feet 4 inches in diameter sunk 30 feet under column 10, 15 feet in diameter sunk 32 feet 8 inches, under column 24, 14 feet in diameter sunk 33 feet 6 inches, under column 25, 10 feet in diameter sunk 33 feet 9 inches—and also made sometimes square, rectangular, or irregular.

The principle is that, as long as the air-pressure in the box is maintained equal to or slightly above the water-pressure upon the outside down to the shoe or lower edge of the caisson, it will be impossible for any water to enter. Work is carried on in the chamber formed by the caisson, in the vast majority of cases, at the same time as the masonry is placed on top.

As the work of excavation advances the caisson sinks, the air-pressure in the inside being reduced slightly until the dead weight of the caisson itself and the masonry upon the top of it are sufficient to overcome the frictional grip or resistance due to the bearing upon the outside surface of the material that it is passing through. In some cases it is necessary to increase the dead load by piling pig-iron on top. Entrance to the caisson is effected through the so-called air-lock; sometimes only one of which is employed and sometimes two, one being for

men and the other for material. This air-lock is a small chamber provided at each end with a door, these doors opening inwardly toward the inside of the caisson. We will suppose that

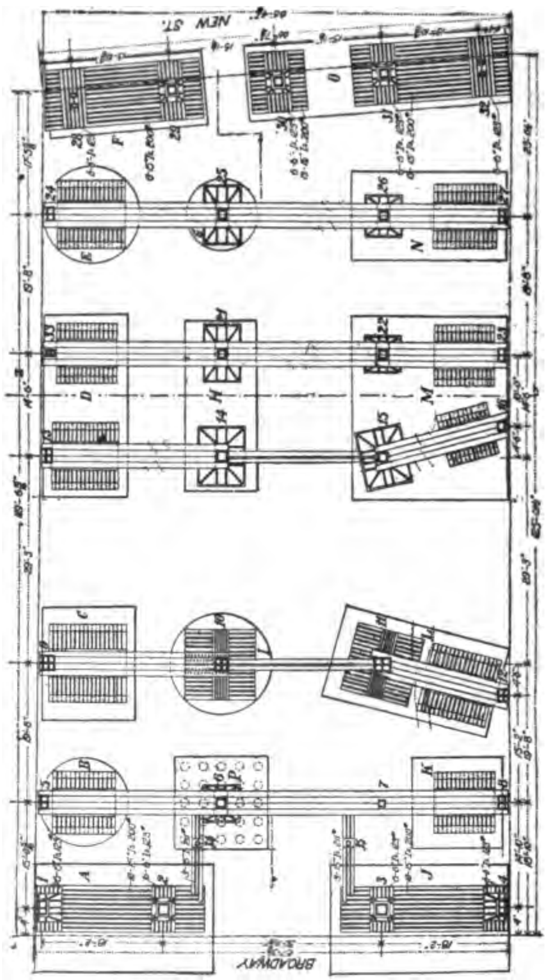


FIG. 109.—PLAN OF CAISSONS AND ARRANGEMENT OF COLUMN BASES.

the inner door of the caisson is closed and the outer door open. The inner door is firmly held in closed position by reason of

the interior air-pressure, which, it is not expected, will at any time exceed 12 or 15 pounds to the square inch, equal to about from 27 to 34 feet head of water. Entering the air-lock the outer door is closed, and the air under pressure admitted through a suitable valve into the air-lock. When the air in the air-lock has become of the same pressure as that in the caisson it is evident that the pressure on the inner door will be equal on both sides and it can be opened, the outer door then preventing the escape of the air under pressure. The reversal of this operation, of course, permits of the air under pressure in the air-lock to escape into the atmosphere.

After the caissons have been sunk to bed-rock, they are cleaned out and filled with concrete, thus forming a continuous pier from the rock up to the surface of the ground.

The fifteen caissons are arranged as shown upon the plan view Fig. 109, and the location of the main columns above them, all but one of which are supported by the caissons. The exception, column No. 6, is carried by 25 piles driven to refusal and capped with a concrete block 10 ft. × 10 ft. × 3 ft. Cylindrical caissons are the most economical and convenient, and would have been used throughout if the conditions had permitted, but the positions of the columns and the necessity of distributing the load along the building lines, and other considerations, determined the use of rectangular ones, except in four cases, *B*, *E*, *I*, and *G*, under columns 5, 24, 10, and 25.

Caisson Detail.—The illustration Fig. 110 represents in detail the construction of caisson *M*, which supports columns 15, 16, 22, and 23. This caisson is 25 ft. 6 in. by 21 ft. 6 in. and 11 ft. 6 in. high. It is built of steel plates, angles, beams, and plate girders. The sides are of ½-inch steel plates, stiffened with angle-brackets made of 6″ × 6″ angles and further strengthened by 7-inch steel bulbs placed horizontally between the brackets. The roof is of ⅝-inch plates, and upon these are placed steel I beams and steel plate girders (see the

sections Fig. 111), to support the loads of masonry while sinking progressed. The sides are carried down a few inches

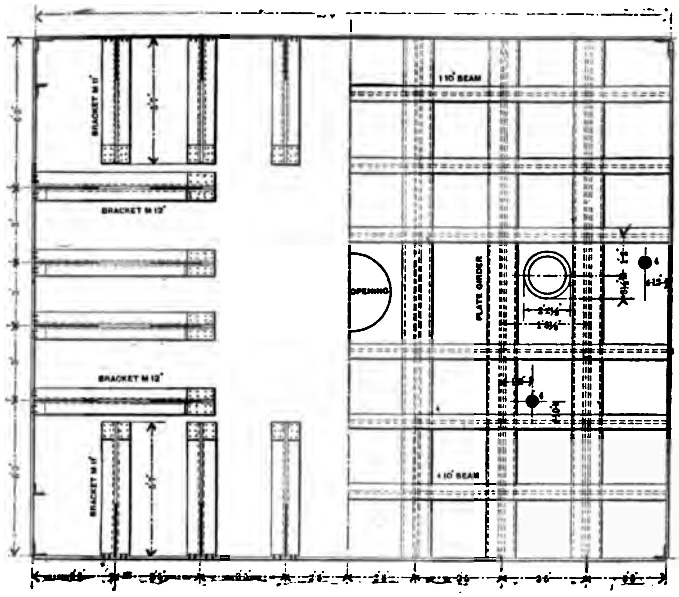


FIG. 110.—SECTIONAL PLAN AND TOP VIEW OF CAISSON *M*.

below the bulbs and the foot of the brackets, and reinforced by heavy steel plates 16 in. wide and $\frac{7}{8}$ in. thick and riveted to

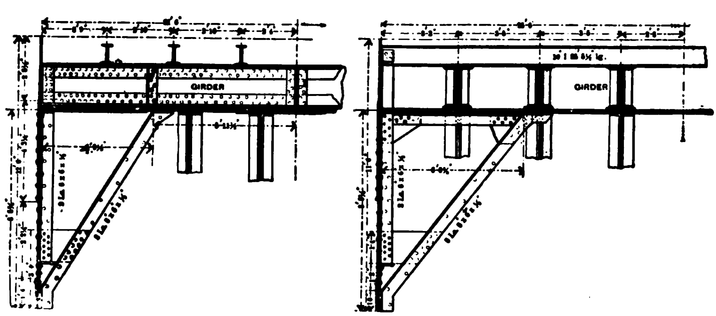


FIG. 111.—CAISSON SECTIONS.

the outer shell with $\frac{7}{8}$-inch countersunk rivets. In the center of roof a shaft 4 ft. in diameter is constructed with air-lock for

the use of men in entering and leaving the working chamber, and also for filling the chamber after the cassion had reached rock. There are also from four to six 4-inch pipes in the roof of each caisson for use as "blowouts" and for the admisions of air.

This caisson contains 467 cubic yards of brickwork and 173 cubic yards of concrete in the chamber.

The construction of the circular caissons is essentially the same as the above.

The substructure contains about 1260 cubic yards of concrete and 3400 cubic yards of brickwork.

Cantilever Construction.—The columns supporting the outer side-walls of the building are located so near the building lines as to be near or beyond the outer edge of the foundation-

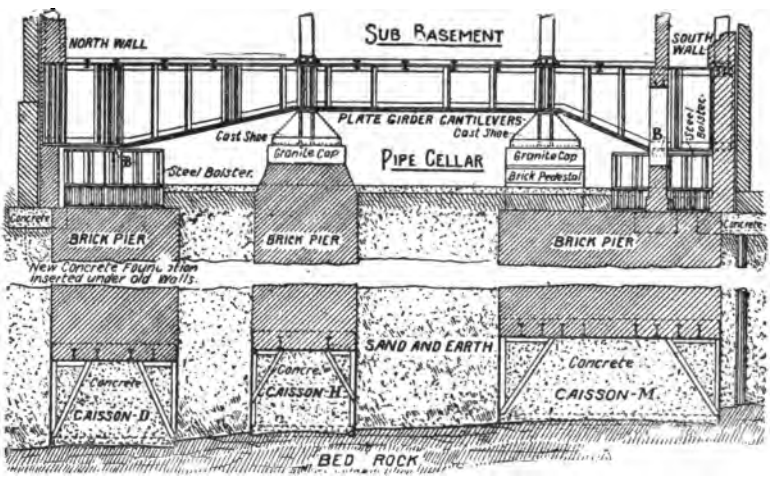


FIG. 112.—TRANSVERSE SECTION OF FOUNDATION AND CANTILEVER GIRDER.

piers, so that if they had been directly supported therefrom they would have loaded it eccentrically and produced undesirable irregularities of pressure. This condition is avoided and the weights transmitted to the centre of the piers by the intervention of heavy plate girders as cantilevers, which support the columns in the required positions and transfer their

loads to the proper bearings above the piers. See the transverse section Fig. 112. From these bearings the load is distributed over the whole area of masonry by special steel bolsters that diminish the unit strains and equalize them throughout. The bolsters as shown under the ends of the cantilever girders consist of a row of plate girders 2 ft. high, and upon these another row at right angles, 3 ft. 6⅝ in. high, rest.

This section also shows the continuous cantilever girders, and the relative location of the three caissons carrying this particular structure.

These cantilever girders consist of a system of plate girders

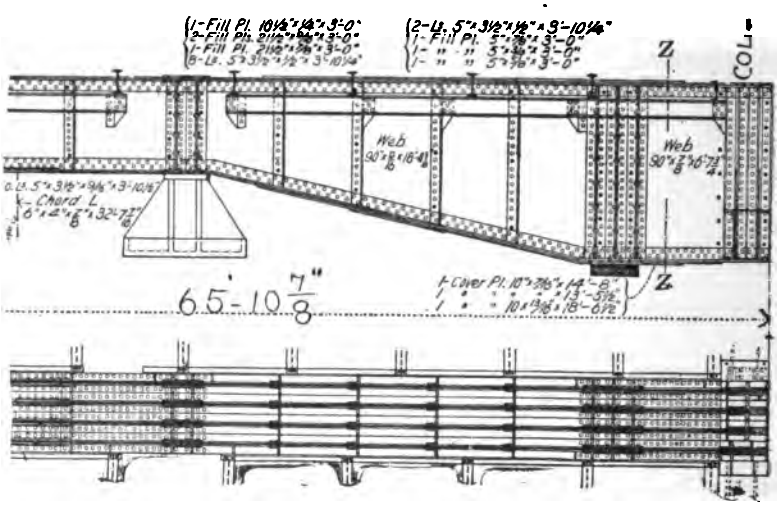


FIG. 113.—CANTILEVER-GIRDER DETAIL.

arranged in a box form as shown in the detail Fig. 113. The height of the girders under centre of column or the bracket part is 6 ft. 7¾ in. It should be particularly noted that the columns at the ends of the cantilevers are on the building line with the exception of space sufficient for the insertion of fireproof bricks. The inner ends of this cantilever are united by

a connecting bridge of plate girders 4 feet deep at columns 21 and 22, columns 23 and 33 being supported at the outer ends. The load supported by the outer columns is transferred to the bolster-shoes at the centre, so that although both of the end columns are outside of the outside edges of their respective caissons, the load they bear is transferred by means of the cantilever and bolster-shoes so as to be evenly distributed over the base of the piers formed by these caissons.

That portion of the specification under details of construction describes in general the conditions upon which this work is constructed. "All stiffeners shown upon the cantilevers are to be $5'' \times 3'' \times \frac{3}{4}''$ steel angles on the inside and $5'' \times 4'' \times \frac{3}{4}''$ angles on the outside. The cantilevers within the building are to rest on steel shoes. The bottom and top bearing surfaces of said shoes shall be planed off perfectly true and level, and that portion resting upon said shoes; and where the columns rest on the cantilevers perfectly level seats shall be prepared as follows:

"A rolled steel plate, one inch thick, of the width of the cantilever in one direction and the width of the steel shoe or the flange of column in the other direction, planed perfectly true, to be riveted to the cantilever with countersunk rivets.

"A solid bearing of the four girders forming a cantilever to be obtained, if necessary, with thin steel plates of such thickness as will bring the bearing surface to a perfectly solid and true contact.

"The tops of the cantilevers to be set level throughout, and the difference in height to be made up in the granite capstone.

"The steel shoe on which cantilevers rest shall be set on the stone caps on a bed formed of heavy sheet lead bedded in Portland cement.

"The girders composing the cantilevers shall be bolted

together with one-inch bolts through and through the stiffener angles; under each line of columns resting on said cantilevers and over the shoe bearing, spaced vertically one foot apart; there will also be two vertical lines of bolts under each column and four lines over the shoe bearing.

"Before setting the cantilever girders in position, the inner sides of the outer girders and both sides of inner girders shall be run full with concrete, and be allowed to set hard."

The first tier of floor-beams are supported by the cantilever girders and so framed as to make the top of beams flush with the top of highest rivet-heads in said girders.

The shoes under cantilevers and the shoes of columns setting directly on granite capstones are made of the best quality cast steel, and free from blow-holes. The cantilevers designed to be placed under the piers and columns of Broadway front are composed as follows:

Two layers of beams,—the lower composed of ten 15-inch steel beams, 200 pounds per yard, running parallel with the front; the upper composed of four beams of the same height and set at right angles to the lower.

Each layer is thoroughly and securely bolted together with separators and bolts, and the spaces between beams are filled with cement.

The failures of the other portions of the work throughout a building due to faulty workmanship are rare in comparison with those due to defective foundations; therefore, a few remarks are inserted on account of the importance the subject bears to the construction of these high buildings, but for fuller information the reader should refer to works which treat solely upon "Masonry Construction."

In designing the foundations of walls and piers when they rest upon a yielding stratum, proper provision must be made for the uniform distribution of the weight, and to form such a

solid base for this superstructure that no movement shall take place after its erection. But all structures built of coarse masonry, whether of stone or brick, will settle to a certain extent; and, with few exceptions, all soils will become compressed under the weight of almost any building.

The main object, therefore, is to proportion the different loads so that the bearing unit of ground area will be equal, and a uniform settlement of the completed structure is ensured.

To Determine the Nature of the Soil.—If the nature of the soil upon which the building is to be constructed cannot be determined by excavations made for surrounding buildings, wells, etc., proper arrangements must be made for testing the subsoil by boring holes at intervals considerably deeper than the walls are intended. It will usually be sufficient to examine the soil with an iron bar, driving it from 4 to 5 feet deeper than the foundation trenches.

In soft soil, a small gas-pipe is driven with a maul from a temporary scaffold, the height of which will depend upon the length of the section of the pipe. Soundings 30 to 40 feet deep can be made in this manner.

Foundations on Rock.—To prepare a rock bed for a foundation, cut away the lower and decayed portions of the rock, and dress it to a plane surface as nearly perpendicular to the direction of the pressure as practicable. If there are any fissures they should be filled with concrete.

The ultimate crushing strength of stone, as determined by crushing small cubes, ranges from 180 tons per square foot for the softest stone to 1800 tons per square foot for the hardest.

The safe bearing power of rock should be about one-tenth of the ultimate strength of cubes; that is to say, the safe-bearing power of solid rock is *not* less than 18 tons per square foot for the softest, and 180 tons for the hardest. Almost any rock when well-bedded will bear the heaviest load than can be brought upon it by any masonry construction.

Foundations upon Clay.—Foundations on clay should be laid at such depths as to be unaffected by the weather; since clay, at even considerable depths, will gain and lose considerable water as the seasons change. The bearing power of clayey soils can be very much improved by drainage or by preventing the penetration of water. When coarse sand or gravel is mixed with the clay, its supporting power is greatly increased, being greater in proportion as the quantity of these materials is greater. When they are present to such an extent that the clay is just sufficient to bind them together, the combination will bear as heavy loads as the softer rocks.

From the experiments made in connection with the construction of the capitol at Albany, N. Y., upon blue clay containing from 60 to 90 per cent of alumina, and the remainder being fine siliceous sand, less than 6 tons per square foot was the extreme supporting power, and 2 tons per square foot the load which might be safely imposed.

The safe load allowed upon ordinary clay if in danger of being saturated by water, is from 1½ to 2 tons per square foot; if kept dry, 3 to 4 tons.

Foundations upon Sand.—Sandy soils vary from coarse gravel to fine sand, and when mixed make one of the best and firmest of foundations. Sand well cemented with clay and compacted, if protected from water, will safely carry 4 to 6 tons per square foot.

Foundations upon Piles.—A pile is generally understood to be a round timber driven into the soil; or, what is called a *bearing*-pile, one used to sustain a vertical load.

Spruce and hemlock answer for foundation-piles in soft or medium soil, or for piles always under water; the hard pines, elm, and beech for firmer soils; the hard oaks for still more compact soils.

The following paragraph from the New York Building Law of 1892 makes provision for the construction of pile and other foundations:

"Every building, except buildings erected upon wharves or piers on the water-front, shall have foundations laid not less than four feet below the surface of the earth, on the solid ground, or level surface of rock, or upon piles or ranging timbers.

"Piles intended for a wall, pier, or post to rest upon shall not be less than *five inches in diameter at the smallest end*, and shall not be spaced more than 30 inches on centres, or nearer, if required by the superintendent of buildings, and they shall be driven to a solid bearing.

"No pile shall be weighted with a load exceeding 40,000 pounds. The tops of all piles shall be cut off below the lowest water-line. When required, concrete shall be rammed down in the interspaces between the heads of the piles to a depth and thickness of not less than 12 inches and for 1 foot in width outside of the piles.

"When ranging and capping timbers are laid on piles for foundations they shall be of hardwood, not less than 6 inches thick, and properly joined together, and their tops laid below the water-line.

"When crib-footings of *iron or steel* are used below the water-level, the same shall be entirely coated with coal-tar, paraffine varnish, or other suitable preparation before being placed in position.

"When footings of iron or steel for columns are placed below the water-level, they shall be similarly coated for preservation against rust." *

"All base-stones shall be well bedded and crosswise, edge to edge. If stepped-up footings of brick are used in place of stone above the concrete, the steps or offsets, if laid in single courses, shall not exceed 1½ in.; or, if laid in double courses, then each shall not exceed 3 in., starting with the brick-work covering the entire width of the concrete, so as to properly distribute the load to be imposed thereon."

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* For foundation-walls and footings, refer to article under New York Building Law relating to skeleton construction, Chapter I.

"If in place of a continuous foundation-wall isolated piers are to be built to support the superstructure, where the nature of the ground and the character of the building make it necessary, inverted arches shall be turned between the piers, at least 12 in. thick and of the full width of the piers, and resting upon a continuous bed of concrete of sufficient area and at least 18 in. thick. Or two footing-courses of large stone may be used, the bottom course to be laid crosswise, edge to edge, and the top course laid lengthwise, end to end; or one course of concrete and one course of stone. The stones shall not be less than 10 in. thick in each course, and the concrete shall not be less than 18 in. thick, and the area of the lower course shall be equal to area of the base-course that would be required under a continuous wall; and the outside pier shall be secured to the second pier with suitable iron rods and plates."

Foundation upon Steel Rails and I-beams.—Steel, usually in the form of railroad rails or I-beams, is used instead of timber in foundations. The rails or I-beams are placed side by side as shown (Fig. 114), and concrete is rammed in between them.

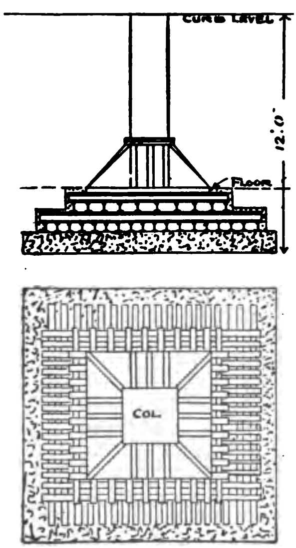

FIG. 114.—STEEL-RAIL FOUNDATION.

The important advantage steel has over wood is that the offset can be much greater, and hence the foundations may be shallow and still not occupy the cellar-space.

The foundation should be prepared by first laying a bed of concrete to a depth of from 4 to 12 in., and then placing upon this a row of I-beams or rails. They should be placed far enough apart to permit the introduction of the concrete filling and its proper tamping between the beams.

Unless the concrete is of unusual thickness, it will not be advisable to exceed 20-in. spacing, since otherwise the concrete may not be of sufficient strength to properly transmit the upward pressure of the beams.

The area of the foundation having been determined and its centre having been located with reference to the axis of the load, the next step is to determine how much narrower each footing-course may be than the one next below it.

The projecting part of the footing resists as a beam, fixed at one end and uniformly loaded.

The load is the pressure on the earth or on the next course below. The offset of such a course depends upon the amount of pressure and the transverse strength of the material.

Evidently the size of beams required will depend upon their strength as *cantilevers* sustaining the upward reaction, which may be regarded as a uniformly distributed load.

Then, for a beam fixed at one end and uniformly loaded,

$$\text{Safe load in lbs.} = \frac{\text{Coefficient}}{4L}.$$

The coefficients for all the different sizes of steel and iron beams are given in Chapter IV, "Floor Loads and Floor Framing."

COMPOUND RIVETED GIRDERS,

AS APPLIED IN THE

CONSTRUCTION OF BUILDINGS.

WITH NUMEROUS

PRACTICAL ILLUSTRATIONS AND TABLES.

BY

WILLIAM H. BIRKMIRE,

AUTHOR OF "ARCHITECTURAL IRON AND STEEL" AND
"SKELETON CONSTRUCTION IN BUILDINGS."

NEW YORK:
JOHN WILEY & SONS,
53 EAST TENTH STREET.

PREFACE.

In order to facilitate the calculation attending the construction of Wrought Iron and Steel Riveted Girders, the author has endeavored in this work to supply the link which separates Theory from Practice. Its object may be briefly stated. A riveted girder is to be designed; the span, depth, and loads are known, the strains are calculated by the well-known bending-moment formulæ, and largely by the graphic method; lastly, the details of construction are fully illustrated.

Touching the question of accuracy, it is scarcely necessary to notice the slight difference that may arise between the two methods, i.e., working out the usual formulæ, or by measuring from the graphic diagrams. The time consumed in wading through a complicated series of equations to reach a few measurements is objectionable when at least such measurements can at once be had by the graphic method.

This work does not investigate exceptional or extremely scientific riveted girders, but more especially those of a type now extensively adopted and constructed by well-known architectural iron workers.

The diagrams and the various examples explaining the Author's method are submitted to architects and architectural students with the hope that they will become a medium of usefulness to them in the routine of office work.

William H. Birkmire.

New York, 1893.

CHAPTER IV.

CAST-IRON LINTELS.

CHAPTER V.

TRUSSES.

CHAPTER VI.

STRUTS.

CHAPTER VII.

CAST-IRON COLUMNS.

CHAPTER VIII.

WROUGHT-IRON COLUMNS.

CHAPTER IX.

STAIRWAYS.

CHAPTER X.

ORNAMENTAL IRONWORK.

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DOORS AND SHUTTERS.

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FLOOR LIGHTS AND SKYLIGHTS.

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HOLLOW BURNT CLAY.

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CHAPTER XVIII.

FINISHING IRON AND STEEL.

CHAPTER XIX.

SPECIFICATION.

CHAPTER XX.

TABLES.

CHAPTER XXI.

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